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APRIL 1958
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JOURNAL
OF THE AMERICAN CONCRETE INSTITUTE

ACI
PROCEEDINGS
VOL. 54

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ACI JOURNAL

Technical pages of the ACI JOURNAL offer a medium for reporting what is new and interesting in concrete research, design, construction, maintenance, or manufacturing. ACI members and JOURNAL readers are urged to share their knowledge and experience by submitting papers and reports for publication in the ACI JOURNAL. Time and changing interests demand that new contributors with new ideas take their place beside the old.

●

The Technical Activities Committee selects the papers, committee reports, discussions, and other contributions for JOURNAL publication. A volunteer corps of experts in various segments of the field assists in technical appraisal of the manuscripts. Technically speaking, acceptable contributions present original material for improvement of design, manufacturing, or construction; confirm, revise, or upset established ideas or practices; or review, digest, or arrange accumulated experience for more ready use.

Further questions raised by TAC: will the contribution fit into a balanced publication schedule? Can it be published at reasonable expense to the Institute?

●

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Secretary, Technical Activities Committee

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JOURNAL
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AMERICAN CONCRETE INSTITUTE

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Coming next month in the JOURNAL

"Tentative Recommendations for Thin-Section Reinforced Precast Concrete Construction" is the report of COMMITTEE 324—Precast Reinforced Concrete, Thin Sections—scheduled for May.

●

FRANK W. CHAPPELL tells how severe structural restrictions were met in the construction of a 265 ft high building of unusual architectural design in his paper, "High-Strength Steel and Concrete Results in Minimum Column Sizes."

●

"Warping of Reinforced Concrete Due to Shrinkage" by ALFRED L. MILLER presents a realistic theory by which the amount of warping can be predicted and provision made for its reduction or elimination.

●

FRITZ KRAMRISCH describes a method used to reinforce the base slab of existing foundations for heavy presses by means of post-tensioning in his paper "Reinforcement of Press Foundations by Post-Tensioning."

●

"High-Density Concrete for Shielding Atomic Energy Plants," by HAROLD S. DAVIS, was originally presented at the ACI regional meeting in Seattle last November.

●

INGE LYSE reports results of tests on two equal beams which were made for the Manda! Bridge in Norway in his paper entitled "Tests of Full-Sized Prestressed Concrete Bridge Beams."

●

"Concept of Elastic Parameters" submitted by VALERIAN LEONTOVICH defines mathematical relations between elastic properties of the member and its elastic parameter.

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Title No. 54-44

ACI's Decade of Progress*

By WALTER H. PRICE†

SYNOPSIS

Retiring President Price evaluates the Institute's success in achieving long-range goals with a look at developments of the past 10 years. Expansion of membership, increased size of the ACI JOURNAL, and heightened technical committee activity are cited, along with progress toward completion of the new ACI headquarters building. Increased Institute activity has been accentuated on the local level through regional meetings and special activities of the technical director, and the Institute's first local chapter has been organized on a provisional basis.

The American Concrete Institute is now 54 years old and we cannot measure our long-range progress by comparing the developments of the last year with the preceding one. I have therefore gone back about 10 years to the time when R. F. Blanks was president, to a report of a special Board committee composed of F. H. Jackson, H. L. Kennedy, M. A. Swayze, H. F. Thomson, and Stanton Walker, with D. E. Parsons serving as chairman. The committee listed eight goals which seemed both desirable and reasonable in October, 1948. Let us see how well we have accomplished those goals, which are listed and discussed below.

1. An increase in membership to approximately double the present membership in 10 years.

In October, 1948, there were 4395 ACI members. Today there are 9223 members. There was an increase of 779 members last year. We are growing at a steady healthy rate and we expect to pass the 10,000 mark next year.

2. Increase in the number of, and improvement in the technical quality of papers with particular emphasis on a larger number of comparatively short papers.

In 1948, an average of 33 papers was published annually in the JOURNAL. Since then, this number has been increased to 54 per year. Beginning in July, 1956, in furtherance of this second goal, the JOURNAL schedule expanded from ten to twelve issues per year. The number of papers being pub-

*Presented at the ACI 54th annual convention, Chicago, Ill., Feb. 26, 1958. Title No. 54-44 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 10, Apr. 1958, *Proceedings* V. 54.

†Member American Concrete Institute, Head of the Engineering Laboratories, U. S. Bureau of Reclamation, Denver, Colo.

lished, however, is limited by the JOURNAL budget, which was increased slightly for the 1958 calendar year. Printing and other expenses have doubled since our last advance in dues, and it appears that consideration of a dues increase may soon be necessary if we are to continue the present service to our members. Papers tend to be shorter, but there is still not a large number of short papers published in the JOURNAL—we have fallen short of this goal.

3. Broaden the scope of technical papers to include more of those describing methods of construction, architectural features, and details of techniques successful in obtaining attractive and durable structures.

Probably less progress has been made toward this goal than any other. Apparently construction people, unlike researchers and designers, are reluctant to write about their experiences. Fortunately, we are reaping the results of construction experiences through some of our technical committee reports and publications. Committee 622, Formwork for Concrete, conducted a survey of forming practices and a summary of the results and tentative recommendations were published in June, 1957. Committee 605, Hot Weather Concreting, published a progress report in the May, 1957 JOURNAL. A revised report of Committee 614, Measuring, Mixing, and Placing Concrete, is now being prepared and should also be of great interest to those who engage in construction. In this connection we should not overlook our recently revised *ACI Manual of Concrete Inspection* and reports of committees on winter protection, curing, painting, application of mortar by pneumatic pressure, and construction of concrete chimneys and silos.

4. Increase in the quantity and scope of technical committee reports.

Committee reports have increased on the average two or three per year over the number published in 1948. Six committee reports were published in the 1954-55 JOURNAL year, 12 in the 1955-56 year, and 10 in the 1956-57 year. Committee reports have been expedited by the appointment in 1954 of a technical director who works closely with the committees in formulating their reports and coordinating the activities of all committees. Active technical committees have increased from 28 in 1948 to 46 in 1957. Six technical committees were established by the Board during 1957, as follows:

116, Nomenclature
201, Durability of Concrete in Service
334, Concrete Shell Structures
335, Deflection of Concrete Building Structures
402, Concrete Floor Finishes

Two committees were reactivated: 322, Design of Structural Plain Concrete, and 805, Application of Mortar by Pneumatic Pressure. Our technical committee structure is strong and I feel certain that the members of the 1948 special committee must be pleased with the progress made toward this goal.

5. Increase in the technical value and the appeal of convention sessions.

Those who have attended conventions over a span of years will, I believe, agree that conduct and content of technical sessions has improved. This is

chiefly due to the Technical Activities Committee's close supervision of the technical program and manner of presentation of papers. Increased staff has also resulted in better control of physical facilities.

6. Increase in the number of and in the importance of discussions of technical papers and committee reports.

Here a few figures will give an idea of how discussion has developed. In the 1947-48 volume year there were 104 pages of published discussion; in 1955-56, 326 pages, and in 1956-57, 246 pages. Discussion is a good indicator of interest in the JOURNAL's contents, and is always welcomed.

Last year, to bring discussion to the reader more quickly, a new policy on publication of discussion was put into effect. You may have noted that your June JOURNAL included discussion of papers published between July and December, 1956; that for January to June, 1957, is found as usual in Part 2, December JOURNAL. This year the transition to quarterly publication will be complete with discussion appearing March, June, September, and December. Under this plan, discussion will reach readers, authors, and committees on a much more current basis.

7. Development of a policy of taking the Institute to the members, such as by increasing the number of regional meetings and the holding of local conferences.

Since the first regional meeting in 1947, regional meetings have been held in ten cities in the United States and Canada. From a modest initial attendance of about 100, they have grown to rival national conventions; in Montreal in 1956, there were 486 present.

This policy was further expanded in 1954 by the addition of a technical director to the ACI staff. One of his responsibilities is arranging local conferences on problems related to concrete. During the winter months he is frequently on the road attending and speaking at these meetings.

A new local chapter which was organized on a provisional basis in the Los Angeles area is the most recent development in the trend toward bringing ACI closer to its members. Revisions of the Institute's Bylaws which were approved for letter ballot vote at this meeting will permit the formal establishment of local chapters. San Francisco and other areas have shown an interest in forming local chapters.

8. Continuing work of the Board's Building Fund Committee looking to ways and means for an ACI headquarters building.

Substantial completion of the new headquarters building evidences the vigor with which the Building Committee under the capable direction of the late Henry L. Kennedy, past president of ACI, carried out this mandate. On January 27 of this year, the Institute moved to the new building from its former crowded and inadequate quarters. This has been made possible by the advanced planning, successful fund raising, and follow-through by the Building Committee. A. Allan Bates now heads that committee, which still has much work to do in arranging for the dedication of the building.

Activity related to the building project has served to bring ACI to the attention of many who were only vaguely familiar with its aims and objec-

tives. The physical structure has already excited much favorable comment and will be a permanent example of the possibilities of imaginative architectural treatment combined with good concrete practice.

As is to be expected, a brief review of ACI activities over the past few years gives a varied picture of accomplishment. Some goals established 10 years ago have been surpassed, while to fully realize others will require continued concentration. Progress to date makes it obvious, however, that ACI is a living, vital organization which owes its success to the efforts of individuals such as yourselves, and to dedicated groups looking into technical problems and methods of getting the information developed into the hands of those who can make practical use of it.

I have received a great satisfaction in the honor you paid me when you elected me president. My work in this office was made easy by the efficient and cooperative help furnished by the headquarters staff. The work of the staff was greatly increased last year by the added responsibility of drafting contracts, buying furniture, and supervising the construction of our new building. These additional duties and the increased volume of society work resulting from our increased membership and publications activities were carried out in a commendable manner.

I have enjoyed my work in ACI and my close associations with so many of its members. I hope to continue this association for many years to come.

Title No. 54-45

Strength of Concrete under Combined Tensile and Compressive Stress*

By DOUGLAS McHENRY† and JOSEPH KARNI‡

SYNOPSIS

An experimental study of the strength of concrete under combined tensile and compressive stresses was undertaken by loading hollow concrete cylinders to failure. Circumferential tension was developed by internal hydrostatic pressure, and axial compression was produced by end loading in a conventional testing machine. Compressive and tensile strengths were both reduced in the presence of an orthogonal stress of opposite sense.

Test results are presented in terms of principal stresses and of octahedral normal and shearing stresses. In neither case does a simple linear relationship between stresses define the failure conditions satisfactorily throughout the full range from simple tension to simple compression.

PURPOSE AND SCOPE OF TESTS

With increasing emphasis directed toward refinements in the design of concrete members on the basis of ultimate strength, the failure characteristics of concrete under combined stress conditions acquire added importance. Considerable work has been reported on the failure characteristics of concrete under triaxial compression with two principal stresses equal, but a very limited amount has been reported for other stress combinations. One of the more important two-dimensional combinations is that of orthogonal tension and compression.

This paper reports a laboratory study of strength of plain concrete subjected to combined tensile and compressive stresses. The results of the investigation are discussed and are analyzed in terms of the octahedral stresses, thus providing a basis for comparing the results with the work of others.

Thin-wall unreinforced concrete cylinders were tested to failure under the action of axial compression applied by a testing machine together with circumferential tension developed by means of internal hydrostatic pressure. The variables included in the series were the stress combinations, the strength of the concrete, and the manner of loading. A total of 91 hollow cylinders of three concrete strengths were tested.

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†Member American Concrete Institute, Director of Development, Portland Cement Association, Chicago, Ill.

‡Member American Concrete Institute, Head of Department of Building Operation Research, Building Research Station, Technion—Israel Institute of Technology, Haifa, Israel; Visiting Engineer, Portland Cement Association, 1954.

TEST SPECIMENS

The unreinforced hollow concrete cylinders were 24 in. high, with 14 in. outside diameter and 10 in. inside diameter. The three different concrete mixes (Table 1) used the same Type I portland cement and the same aggregates. Nominal design strengths for 6 x 12-in. cylinders fog cured for 28 days were 3000, 5000, and 6500 psi for mixes designated A, B, and C, respectively.

TABLE I—MIX DATA

Mix	A 3000-psi concrete		B 5000-psi concrete		C 6500-psi concrete	
Sand, percent by weight						
Elgin	42.3		38.3		33.8	
Lake	11.0	53.3	9.9	48.2	8.7	42.5
Gravel, percent by weight						
4—⅜	21.3		22.6		25.5	
⅜—¾	25.4	46.7	29.2	51.8	32.0	57.5
		100		100		100
Water-cement ratio by weight		0.77		0.56		0.41
Cement:sand:gravel ratio by weight		1:4.6:4.0		1:2.9:3.1		1:1.9:2.5

Some specimens were cast in steel molds and others were cast in waterproof paper molds, but no difference in strength characteristics was noted. All specimens were consolidated by internal vibration.

Three hollow cylinders and four 6 x 12-in. control cylinders were prepared from each batch. The specimens were cured in the molds 24 hr and were then transferred to the moist room (100 percent relative humidity, 73 F) for 27 days. The steel molds were stripped at the time of transfer, but the paper molds were not removed until 1 or 2 days later. Both ends of the hollow cylinders were capped with neat cement paste while the specimens were in the moist room. All specimens were tested at 28 days during the first 15 min after removal from the moist room.

TESTING PROCEDURES

Manner of loading

The longitudinal compression was applied by a 1,000,000-lb testing machine equipped with a spherical-seated head. Internal hydrostatic pressure was developed by a water-filled rubber cylinder, which was sealed and inserted into the hollow concrete cylinders. A rubber membrane was placed on top of the rubber cylinder, and the entire assembly was covered with a machined steel plate through which hydraulic pressure was applied from a hand pump into the rubber membrane, as shown by Fig. 1. The pressure exerted against the head of the testing machine was compensated by bringing the head into a position just contacting the steel plate, and measuring the upward force acting against the weighing mechanism of the testing machine. The load observed on the testing machine was checked against the pressure gage in the hydraulic system, and this pressure was maintained constant in most of the tests.

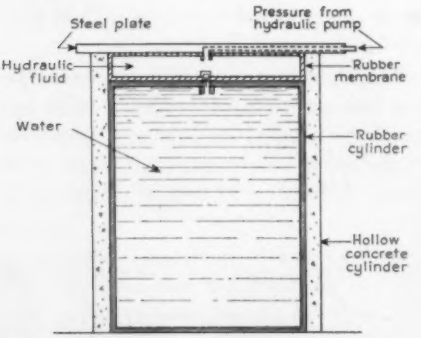


Fig. 1—Method of applying the internal hydrostatic pressure

In Series A, B, and C the internal hydrostatic pressure was applied to a predetermined value, and the axial load was then increased until failure occurred. During the increase in the axial thrust, the internal hydrostatic pressure was kept constant.

In Series AA, conducted on some of the 3000-psi concrete specimens, the axial load was first applied to a predetermined value, and the internal hydrostatic pressure was then increased until failure occurred. No significant differences were observed in the results of Series AA and Series A.

Test program

Table 2 outlines the test program. In addition to the 91 hollow cylinders, conventional 6 x 12-in. control cylinders were tested for each batch to provide a basis of comparison among the different tests.

TABLE 2—OUTLINE OF TEST PROGRAM

Type of loading	Series			
	A	AA	B	C
17 specimens failed under axial thrust only	3	—	4	10
13 specimens failed under internal pressure only	3	—	4	6
61 specimens failed under combinations of internal pressure and axial thrust	27	6	11	17
Total (91 specimens)	33	6	19	33

Series A and AA are treated as one group in presenting the data; accordingly, Mix A is represented by 39 specimens, B by 19 specimens, and C by 33 specimens. Most tests were conducted in triplicate.

ANALYSIS OF RESULTS

Adjustment of test results for comparison

Stresses at failure of the hollow cylinders and of the corresponding control cylinders are listed in Table 3. Because of variations in the strength of the concrete between different batches, the results are adjusted to permit comparison on an equivalent basis. Each stress is multiplied by the ratio of the strengths of the control cylinders, using as a base the control cylinders cast with the hollow cylinders tested under pure thrust. Thus, for pure thrust in

Series A and AA	$f_c' = -3150$ psi
Series B	$f_c' = -5270$ psi
Series C	$f_c' = -6360$ psi

where f_c' is the unconfined compressive strength of the control cylinders.

The compressive strength of the hollow cylinders varied from 75 to 85 percent of the strength of the solid control cylinders, and the percentage increased with strength. The maximum tensile stresses in the hollow cylinders ranged from 6 to 9 percent of the compressive strength of the solid cylinders (7 to 12 percent of maximum compression in the hollow cylinders), and the percentage decreased with increasing strength.

TABLE 3—PRESENTATION OF DATA

f'_c, strength of control cylinders, − psi	Actual stresses		Adjusted stresses		Percent of tensile strength	Dimensionless ratio	
	Compression − psi	Tension + psi	Compression $-\sigma_3$, psi	Tension $+\sigma_1$, psi		$+\frac{\sigma_3}{\sigma_c}$	$-\frac{\sigma_1}{\sigma_c}$
Series A: nominal 6 x 12-in. cylinder strength 3000 psi							
3150	2380	0	2380	0	0	1.00	0.
3030	2270	46	2360	48	17	0.99	0.020
3180	2360	77	2338	76	26	0.98	0.032
3140	2130	122	2137	122	42	0.90	0.051
3070	1760	135	1806	139	48	0.76	0.058
3370	1600	164	1496	153	53	0.63	0.064
2910	350	191	379	207	72	0.16	0.087
3210	660	214	648	210	73	0.27	0.088
3370	440	236	411	221	77	0.17	0.093
3140	128	236	120	237	82	0.05	0.100
3150	0	288	0	288	100	0.	0.121
Series AA: nominal 6 x 12-in. cylinder strength 3000 psi							
2830	2020	95	2248	106	37	0.94	0.045
3070	1520	153	1560	157	55	0.66	0.066
3070	300	242	308	248	86	0.13	0.104
2830	130	247	145	275	95	0.06	0.116
Series B: nominal 6 x 12-in. cylinder strength 5000 psi							
5270	4300	0	4300	0	0	1.00	0.
5410	3970	76	3867	74	21	0.90	0.017
4910	2380	127	2555	136	38	0.59	0.032
5500	2910	144	2788	138	39	0.65	0.032
4910	2470	175	2651	188	53	0.62	0.044
4910	440	207	472	222	62	0.11	0.052
5150	360	232	368	237	66	0.09	0.055
5270	0	358	0	358	100	0.	0.083
Series C: nominal 6 x 12-in. cylinder strength 6500 psi							
6360	5430	0	5430	0	0	1.00	0.
6530	5220	127	5084	124	33	0.94	0.023
6530	5310	159	5172	155	41	0.95	0.029
6000	3700	199	3922	211	55	0.72	0.039
6390	2750	207	2737	206	54	0.50	0.038
5750	870	208	962	230	60	0.18	0.042
7250	1190	323	1044	283	74	0.19	0.052
7000	820	326	745	296	78	0.14	0.055
7000	380	380	345	345	91	0.06	0.064
6340	0	380	0	380	100	0.	0.070

Interpretation of results

Biaxial relationships between combined stresses on the hollow cylinders at failure for the tests from all three mixes are shown in Fig. 2. For greater clarity, the compressive scale on the graph is five times the tensile scale. The tensile stress produced by the internal hydrostatic pressure has been assumed uniform across the wall of the cylinder in each test. This assumption was adopted because of the likelihood that inelastic action at the approach of failure would minimize any effect of nonuniformity.

In the analysis of the test results, σ_1 is the adjusted tensile stress determined by the internal hydrostatic pressure by the formula for thin-wall cylinders, σ_2 is the adjusted radial stress from the internal hydrostatic pressure (neglected in the biaxial analysis), and σ_3 is the adjusted compressive stress from the thrust placed on the ends of each specimen. Tension is selected as positive and compression as negative. Algebraic signs are shown in column headings of the tables together with the units of measurement.

The column headed "percent of tensile strength" in Table 3 gives the ratio of adjusted tensile stress at failure on each hollow cylinder to ultimate stress on the hollow cylinder of the same mix proportions tested under hydrostatic pressure only.

The dimensionless ratios are computed from adjusted stresses by dividing by σ_c, strength of the hollow cylinders of the same mix in pure thrust. These results are shown in Fig. 3 from data in the last two columns of Table 3.

Types of failure

Fig. 4 shows the gradual transition from "compression type" failures to "tension type" failures as percent of tension was increased.

In most cases failures of the hollow cylinders were very sudden, and the "compression type" failures of the 5000-psi and 6500-psi specimens were almost explosive.

The manner of loading in Series AA as compared with Series A produced no difference in type of failure; rather, the governing factor was the percent of tension applied in the test.

Octahedral stresses

It is apparent from Fig. 3 that the failure criterion for these tension-compression combinations cannot be formulated in terms of a simple direct

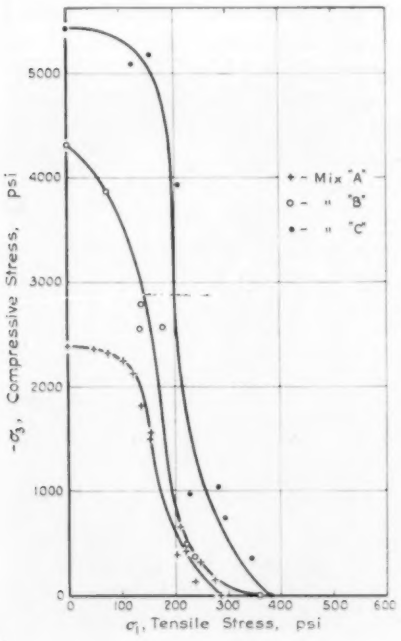


Fig. 2—Relationships between combined stresses at failure

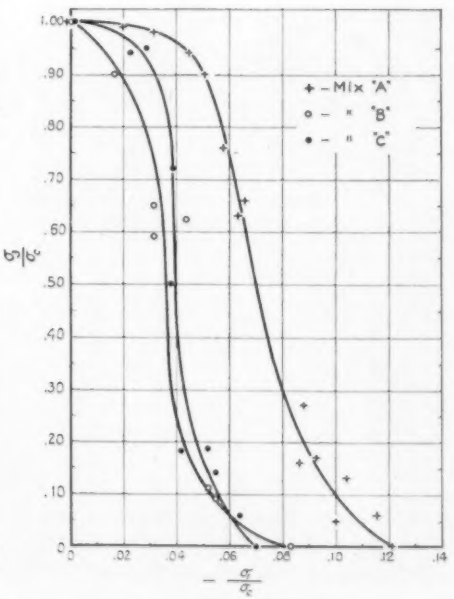


Fig. 3—Relationships between combined stresses at failure when expressed as dimensionless ratios

TABLE 4—OCTAHEDRAL STRESSES

Normal stress $-\sigma_0$, psi	Shearing stress $+\tau_0$, psi	Dimensionless ratios $+\frac{\sigma_0}{\sigma_c}$	Dimensionless ratios $-\frac{\tau_0}{\sigma_c}$
Series A: nominal 6 x 12-in. cylinder strength, 3000 psi			
792	1122	0.333	0.471
771	1124	0.324	0.472
754	1120	0.317	0.471
672	1037	0.282	0.436
556	886	0.234	0.372
448	744	0.188	0.313
57	243	0.024	0.102
146	365	0.061	0.153
63	262	0.026	0.110
−39	148	−0.016	0.062
−96	136	−0.040	0.057
Series AA: nominal 6 x 12-in. cylinder strength 3000 psi			
714	1086	0.300	0.456
468	775	0.197	0.326
20	227	0.008	0.095
−43	174	−0.018	0.073
Series B: nominal 6 x 12-in. cylinder strength 5000 psi			
1433	2027	0.333	0.471
1264	1841	0.294	0.428
806	1238	0.187	0.288
883	1348	0.205	0.313
821	1296	0.191	0.301
83	289	0.019	0.067
44	249	0.010	0.058
−119	169	−0.028	0.039
Series C: nominal 6 x 12-in. cylinder strength 6500 psi			
1810	2560	0.333	0.471
1653	2426	0.304	0.447
1672	2475	0.308	0.456
1237	1901	0.228	0.350
844	1341	0.155	0.247
244	516	0.045	0.095
254	571	0.047	0.105
150	438	0.038	0.081
0	282	0.	0.052
−127	180	−0.023	0.033

relationship between the principal stresses. The shape of the curves indicates a polynomial of fourth degree or higher, or a relationship of the form

$$\sigma_3 = A^b \sigma_1^c$$

The Mohr theory of failure, which has been widely used for conditions of biaxial and triaxial compression, does not appear to be applicable. The curves of Fig. 3 lead to Mohr envelops of complex shape which seem to have little or no physical significance.

Among various theories of strength which have been generally recognized, only one appears useful in defining the surface of failure* in the present case. That one is the theory that octahedral shearing stress at failure is a function of the octahedral normal stress.† Bresler and Pister[1] investigated the applicability of that criterion to concrete failing in combined compression and torsion, and reported a linear relationship within the range of their test data.

The octahedral[2] normal stress σ_0, and shearing stress τ_0, are computed from the adjusted data of Table 3 by the formulas

$$\sigma_0 = \frac{\sigma_1 + \sigma_2 + \sigma_3}{3} \quad \ldots\ldots(1)$$

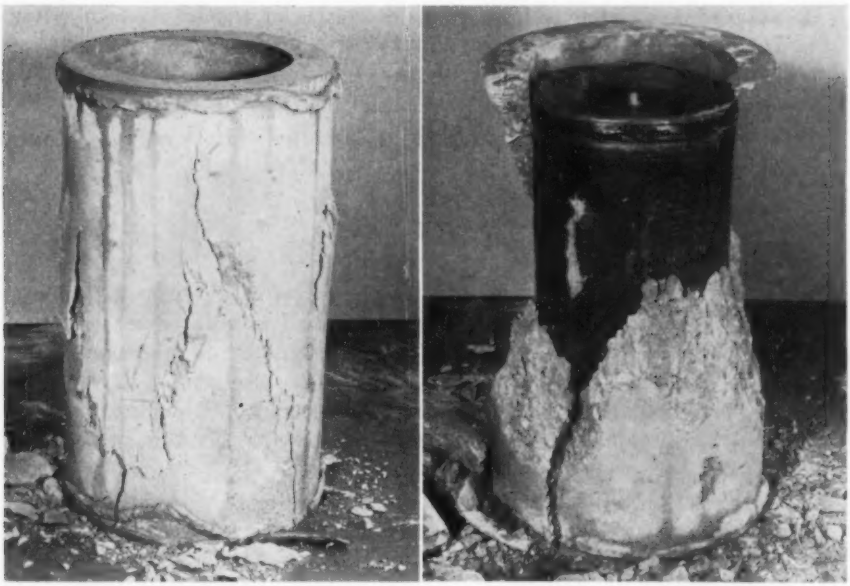
and

$$\tau_0 = \tfrac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2} \quad \ldots\ldots(2)$$

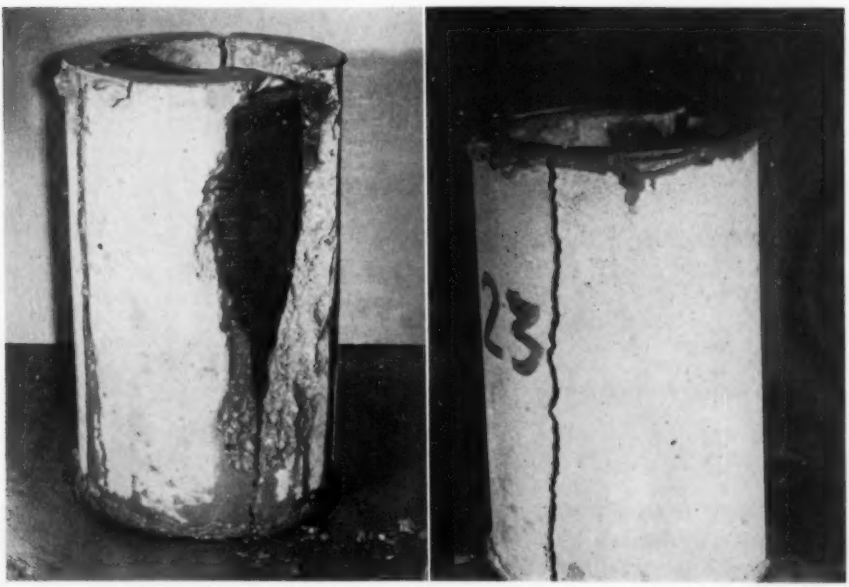
As these experiments represent essentially a condition of biaxial stress, i.e., the intermediate principal stress is small in comparison with maximum stress, σ_2 has been taken as zero throughout the computations in Table 4.

*The surface of failure is a theoretical bound in stress space within which any principal stress combination is admissible. Stress combinations which fall on or outside this bound are inadmissible in that the material is incapable of maintaining integrity under such a state of stress.

†The closely related criterion used by Freudenthal[2] involving the "effective stress," which is related to the octahedral shearing stress through a constant, would doubtless be equally useful.



16 percent tension 27 percent tension



47 percent tension 82 percent tension

Fig. 4—The transition from compression to tension failure

Relationships between the dimensionless ratios of the octahedral stresses from Table 4 are shown by the solid lines in Fig. 5, 6, and 7 for Mixes A, B, and C, respectively. Graphically the results are essentially linear for all three mixes except for the curvature at extremes of the data.

From elastic analysis, radial stress at the internal surface of the cylinders is given by

$$\sigma_2 = -\frac{t}{r}\sigma_1 = -\frac{2}{5}\sigma_1$$

in which r is the internal radius and t is wall thickness. The value of σ_2 reduces to zero at the external surface. Points from the triaxial analysis computed from maximum values of σ_2 are shown on the graphs as triangles (Δ) in contrast to the other points computed from $\sigma_2 = 0$. The general nature of the relationship of octahedral stresses at failure is altered only slightly by introducing the maximum value of σ_2. The radial stress is therefore neglected in the following discussion.

A linear relationship between octahedral stresses may be derived from the terminal conditions at failure. Thus, from Eq. (1) and (2) we obtain:

for simple compression ($\sigma_1 = 0, \sigma_2 = \sigma_c$)

$$\frac{\sigma_0}{\sigma_c} = \frac{1}{3} \qquad \frac{\tau_0}{\sigma_c} = \frac{-\sqrt{2}}{3}$$

for simple tension ($\sigma_1 = \sigma_t, \sigma_2 = 0$)

$$\frac{\sigma_0}{\sigma_c} = \frac{1}{3}\,\frac{\sigma_t}{\sigma_c} \qquad \frac{\tau_0}{\sigma_c} = \frac{\sqrt{2}}{3}\,\frac{\sigma_t}{\sigma_c}$$

The connecting line is then given by

$$\tau_0 = \frac{2\sqrt{2}}{3}\,\frac{\sigma_c\ \sigma_t}{\sigma_c - \sigma_t} - \sqrt{2}\,\frac{\sigma_c + \sigma_t}{\sigma_c - \sigma_t}\sigma_0 \qquad (3)$$

which is in the desired form,

$$\tau_0 = a + b\,\sigma_0$$

This linear relationship in the dimensionless form is shown for each of the three mixes by broken lines on Fig. 5, 6, and 7. It is apparent that the experimental points fall very close to the straight line. A consistent deviation is also apparent, indicating that experimental data define a curve which is slightly S-shape, rather than a straight line. Data given by Bresler and Pister[1] for the range $0 < \sigma_0/\sigma \leqq \frac{1}{3}$ suggest the same S-shape.

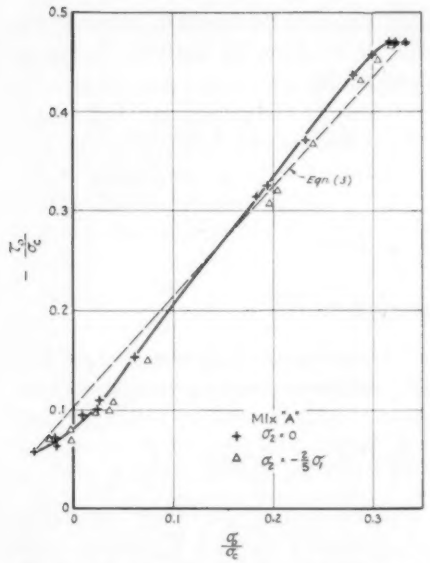


Fig. 5—Relationship between octahedral stresses at failure for Mix A

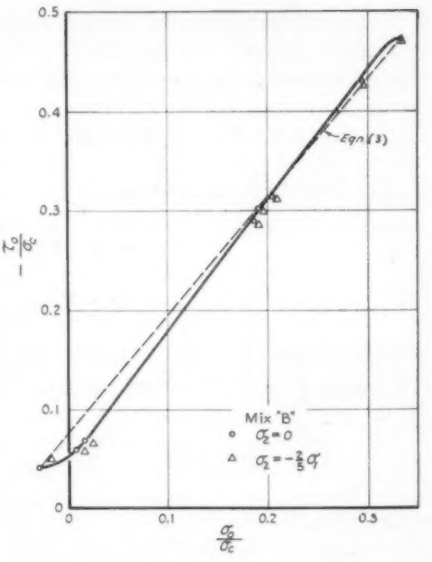


Fig. 6—Relationship between octahedral stresses at failure for Mix B

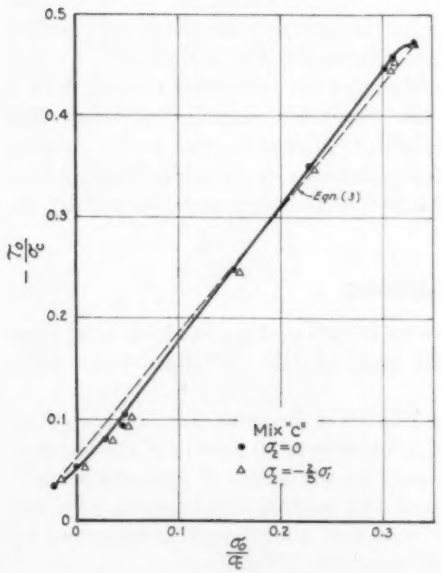


Fig. 7—Relationship between octahedral stresses at failure for Mix C

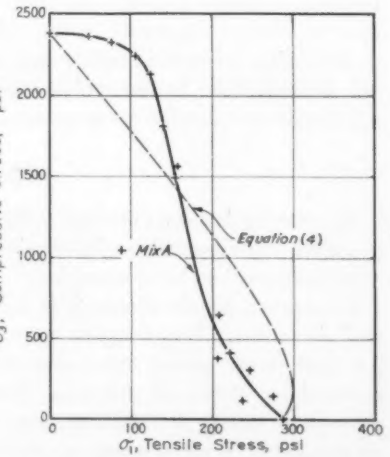


Fig. 8—Comparison of Eq. (4) with experimental data

Substitution of Eq. (1) and (2) into (3) yields a relationship between the principal stresses corresponding to the linear relationship between the octahedral stresses. For $\sigma_2 = 0$ the relation is given by

$$(b^2 - 2)(\sigma_1^2 + \sigma_3^2) + 2(b^2 + 1)(\sigma_1\sigma_3) + 6ab(\sigma_1 + \sigma_3) + 9a^2 = 0$$

For Mix A, as an example,

$$\sigma_c = -2380 \qquad \sigma_t = 288$$

$$a = 242 \qquad b = -1.109$$

The equation $\tau_0 = 242 - 1.109\sigma_0$ after dividing by σ_c yields the straight line plotted in Fig. 5. The corresponding relationship between the principal stresses, given by

$$0.77(\sigma_1^2 + \sigma_3^2) - 4.46\,\sigma_1\sigma_3 + 1610(\sigma_1 + \sigma_3) - 527{,}076 = 0 \ldots\ldots\ldots (4)$$

is plotted in Fig. 8 together with the experimental curve. The conversion from a linear relationship in the octahedral stresses yields a hyperbolic relationship in the principal stresses which does not adequately fit the original data.

A generalized criterion of failure expressed as a simple relationship between two of the stress invariants, such as the above relationship between the octahedral stresses, would indeed be useful, especially for triaxial stress combinations. However, it appears that a surface of failure defined by such a relationship might permit as a close approximation some stress combinations which in terms of principal stresses would be found to be strictly inadmissible.

At the present stage of our knowledge the accumulation of test data is perhaps of more importance than is the search for simplified criteria. The present data indicate that the relationship between σ_1 and σ_3 for tension-compression failures is rather complex and is not well defined. Further tests are needed, also, to determine whether this relationship is influenced by the particular test conditions employed.

CONCLUSIONS

By testing hollow concrete cylinders to failure under combined axial compression and circumferential tension the range of data for failure under stress combinations has been extended.

In general, tensile strength is reduced by an orthogonal compressive stress, and vice versa. Very roughly, when either stress is at 50 percent of the ultimate for that stress acting alone, the resistance to the stress of opposite sense is reduced by about 50 percent. However, the relationship between principal stresses at failure is nonlinear, and for these test conditions is represented by an elongated S-curve whose shape is not precisely defined.

The principal stress relationship at failure appears to be independent of the sequence in which the stresses are applied.

The Mohr theory of failure is not applicable in the tension-compression range.

An essentially linear relationship was found between the octahedral normal stress and the octahedral shearing stress at failure except for data near the conditions of simple compressive and simple tensile failure. However, a linear relationship over the full range of the data was found to be unacceptable.

The need for a generalized failure criterion is recognized, but it is believed that for the present the limiting conditions for tension-compression combinations must be based on actual test values.

ACKNOWLEDGMENTS

The investigation herein described was conducted at the Research and Development Laboratories of the Portland Cement Association. Consistent test data were achieved only through careful planning and attention to all details of fabricating and testing the specimens. Acknowledgment is made to J. R. Janney, consultant, Chicago, for advice and assistance in that respect. Credit is due also to G. G. Balmer, development engineer, PCA, for assistance in computation and interpretation of the test results. A considerable study was made of the various theories of strength of materials, but for the most part they were found not applicable to the present case.

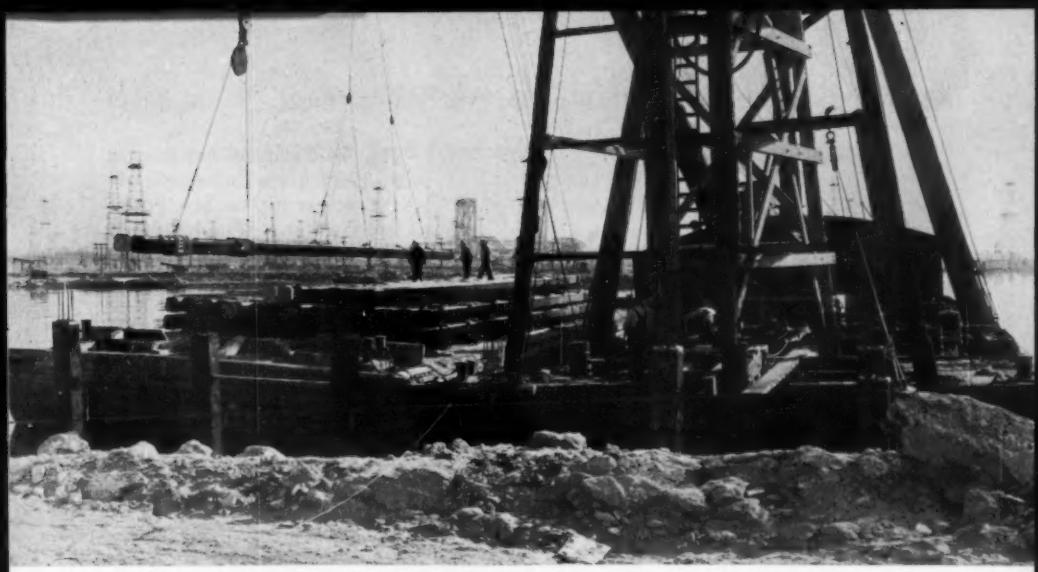
REFERENCES

1. Bresler, B. and Pister, K. S., "Failure of Plain Concrete under Combined Stresses," *Proceedings*, ASCE, V. 81, Separate No. 674, Apr. 1955.

2. Freudenthal, A., "The Inelastic Behavior and Failure of Concrete," *Proceedings*, First U. S. National Congress of Applied Mechanics, 1951, pp. 641-646.

3. Nadai, A., *Theory of Flow and Fracture of Solids*, McGraw-Hill Book Co., Inc., New York, 1950.

Discussion of this paper should reach ACI headquarters in triplicate by July 1, 1958, for publication in the Part 2, December 1958 JOURNAL.



Multiple-point lifting device unloads precast, reinforced, asphalt impregnated concrete piles from a barge at Los Angeles harbor

Title No. 54-46

Use of Concrete in Marine Environments*

By C. M. WAKEMAN,† E. V. DOCKWEILER,‡ H. E. STOVER,§ and L. L. WHITENECK**

SYNOPSIS

Results of service tests indicate use of properly engineered concrete in marine environments is feasible, economical, and presents no unusual problem to the engineer. Discussion of this paper centers around several theories for the deterioration of concrete in sea water and concludes with recommendations for correct procedures to be observed for marine concrete construction.

INTRODUCTION

Should concrete piles and concrete wharf decks be used in new pier construction?

Will concrete endure in sea water, or will it eventually constitute a built-in maintenance problem?

Why do some concrete structures seem to last indefinitely in sea water, while others deteriorate within a comparatively short time? What has been

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†Member American Concrete Institute, Testing Engineer, Los Angeles Harbor Department, Los Angeles, Calif.
‡Rear Admiral, U. S. Navy, Retired, Chief Harbor Engineer, Los Angeles Harbor Department, Los Angeles, Calif.
§Assistant Testing Engineer, Los Angeles Harbor Department, Los Angeles, Calif.
**Vice-President, Plicoflex, Inc., Los Angeles, Calif.

Fig. 1—Concrete blocks cast in 1905 were submerged in the ocean in the San Pedro breakwater. After inspection in 1932, the blocks were again submerged in their original locations where they remain today

done to the concrete to make the successful structures endure? Just what is the life expectancy of concrete in sea water?

These are but a few of the many questions which the average engineer having limited experience with marine construction might ask. This paper has been prepared as a review which will answer some of these questions on the basis of the authors' long experience with concrete in sea water, at the same time acknowledging and citing the work of others.

Concrete can be successfully placed under water, above water, or between tides. Recommendations herein emphasize the necessity of proper proportioning and good workmanship in order to provide the dense, impervious concrete, having a minimum permeability and water absorption characteristics so necessary for endurance. Just how this is to be accomplished is not within the province of this report; it has been comprehensively and adequately covered in a recent Corps of Engineers report[28] as well as in the ACI standard "Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-42)."[35]

EARLY MARINE INSTALLATIONS—LOS ANGELES

Some of the earliest recorded concrete used in marine construction in the Los Angeles harbor was cast in 1905, when D. E. Hughes, United States Army engineer, supervised the making of sixteen mass concrete blocks (approximately 4 x 5 x 5 ft), of various mixes with different brands of cement. These blocks were placed in a submerged position in the San Pedro Breakwater in 1906, where they remained until 1932.[27,28] At this time they were removed (Fig. 1) and placed on a barge for inspection. Cores were taken and submitted to the National Bureau of Standards for a complete analysis. A significant statement[9,49] was made by the then acting director of that bureau, Lyle Briggs, to the effect that, ". . . [tests of several cores] . . . would seem to indicate that some magnesia from the sea water has replaced part of the lime of cement. This fact may be solely due to the question of sampling.

There apparently has been no increase in the SO_3 content which is what should accompany this [form of] disintegration. . . .

"The physical test data would seem to indicate that at least in the sample received there has been no apparent disintegration of the concrete."

The writers observed and photographed the blocks when they were removed in 1932. These specimens were returned to their original locations where they remain to this day. A diver's inspection in 1955 revealed that after 50 years' service in sea water, all edges and corners of the blocks remain hard, sharp, and well defined and the surfaces showed no physical signs of deterioration. Even the impressions of the form boards were plainly visible. Several long concrete bulkhead walls in the Los Angeles Harbor (bases under water but tops exposed at high tide) are in equally good condition after approximately the same service life.

Thousands of precast, reinforced concrete piles driven between 1910 and 1914 at Los Angeles harbor are still in service. The submerged portion of all of these piles is in excellent condition. Some have been repaired in the area from the tidal zone to the pile tops because longitudinal cracks over the reinforcing steel caused spalling of the outer surface.

ASPHALT IMPREGNATED PILING

The Los Angeles Harbor Department pioneered in the practice of waterproofing precast concrete piling[32] (beginning in 1923) by impregnating the outer shell of the concrete to a depth of from ¾ in. to 2½ in. with asphalt (Fig. 2, 3). This was accomplished by vacuum-pressure methods similar to the treatment of creosoted wood. After pre-drying at temperatures up to 250 F, the concrete is placed in a closed retort in a bath of molten asphalt admitted under a high vacuum, then subjected to air pressure up to 150 psi, with temperature gradients being carefully controlled throughout the entire treatment. Concrete is held several hours at 100 psi pressure after asphalt is removed, then is moved to an enclosed cooling chamber. This process is

Fig. 2—A plant at Wilmington, Calif., for the manufacture of precast, asphalt impregnated, reinforced concrete piling. Untreated piles are shown on cars at left, about to enter the preheating chamber. Next in line are the asphalt pressure tank and cooling sheds. Finished product lies on the ground to right

Fig. 3—Cross section of test specimens impregnated with asphalt. On left, a special porous concrete used as a 2½-in. outer protective coating on concrete piles between elevations of −4 and +17 ft. Note complete penetration of asphalt, compared with specimens on the right of structural grade concrete, which allowed only partial impregnation

applicable to precast concrete units of all kinds which fit into suitable pressure chambers (6 to 8 ft in diameter by 70 to 100 ft long).

Thousands of bearing piles of this type have been in successful use in Los Angeles Harbor and elsewhere, since their inception in 1925, with no indication of deterioration to date. Characteristics of typical concrete used in the piles are shown in Table 1. Asphalt impregnated beams and cylinders, which have been stored in the ocean at an elevation of −35 ft have been found to be bone dry when tested 30 years later.

SPECIAL CONCRETING TECHNIQUES

Shotcrete

A mixture of portland cement, sand, and water which is sprayed onto a half form is variously termed "shotcrete," "gunite," "spraycrete," etc. This

TABLE 1—AVERAGE CHARACTERISTICS OF TYPICAL CONCRETE USED IN ASPHALT TREATED PILING (LAHD REPORT NO. 2554)

Mix—1:3:3¼	Average compressive strength at time of treating:
Slump—1¾ in.	Untreated....2925 psi
Average asphalt penetration into concrete, ½ in.	Treated....2640 psi
Average water absorption of concrete:	Average transverse strength at time of treating:
Untreated....6.40 percent	Untreated.... 630 psi
Treated....None	Treated.... 780 psi
Average compressive strength at 28 days:	Average modulus of elasticity at 1500 psi:
Untreated....3160 psi	Untreated....3,150,000 psi
Average transverse strength at 28 days:	Treated....3,250,000 psi
Untreated.... 760 psi	Average bond strength:
	Untreated.... 345 psi
	Treated.... 300 psi

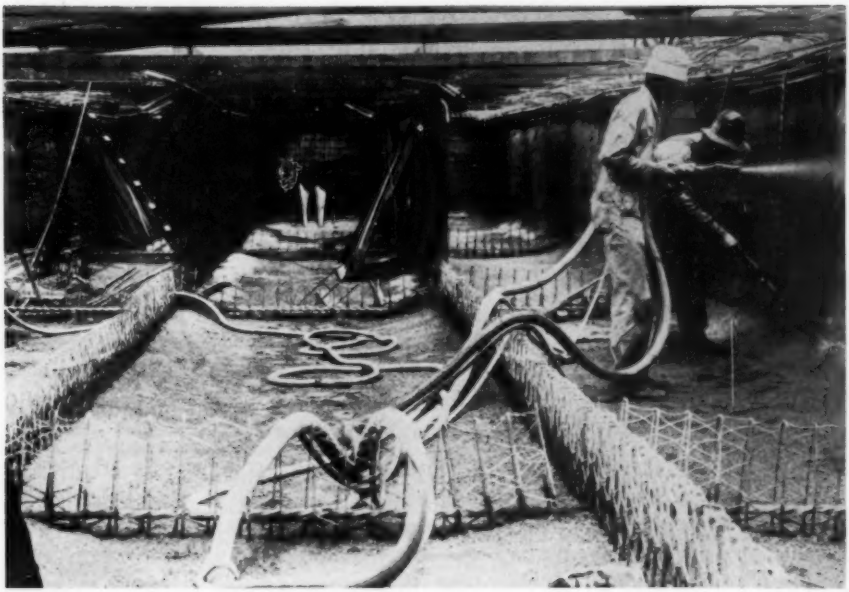

Fig. 4—Shotcreting a concrete landing barge in 1918. When bottom and sides had been "shot" as shown, and bulkheads were in place, the top deck was likewise fabricated with shotcrete

product has been successfully used in marine construction for many years in piling, building walls, beams, concrete barges, etc. A large landing barge (Fig. 4) for a passenger ferry terminal was fabricated entirely with reinforced shotcrete and launched in the year 1919. This unit is still in service after 38 years in sea water,[27] and has required no maintenance.

Both hollow and solid shotcrete piles (Fig. 5, 6) have been in sea water service for about the same time as the barge in the same harbor. All solid shotcrete piles were transported to point of installation in vertical racks; they were never handled horizontally, thus eliminating bending stresses. Shotcrete jackets for wood piles have likewise been used for years and are described elsewhere.

Shotcrete concrete for marine usage is usually proportioned in the ratio of one part of cement (by volume) to four parts of well graded sand, although occasionally an engineer may prescribe mixtures as rich as 1:3 or even 1:2.75. Rich mixes are usually inadvisable, as they increase the tendency to cracking by reason of stresses caused by temperature variations and alternate wetting and drying.[27] Average 28-day compressive strengths of 4000 psi may be obtained with 1:4 mixes, which is sufficient strength for most purposes.

One advantage of using shotcrete over ordinary concrete, is that it effects considerable saving in material and labor. Also shotcrete is more uniform and dependable than conventionally placed concrete if field control of placing

is limited or inadequate. It has a greater bond to old concrete surfaces than cast concrete and is useful for patching and plugging. Its limitations are that its placement is more critical than cast concrete and that it requires extraordinarily conscientious, careful, and experienced craftsmen. In one respect the process of shotcreting may be likened to the welding of metal. Both depend for good results, to a pronounced degree, upon the skill of the operators. The shotcrete operator must constantly regulate the consistency of the mix as the raw materials are discharged from the nozzle. He must know exactly how much pressure to use and how far to stand from his work. The angle of the nozzle with respect to the work is critical, as is the rate of application, so that a porous structure will not be created due to the rebound of part of the aggregate. Other disadvantages are that shotcrete is not well adapted to the construction of massive members and that reinforcing steel must be secured with more care than is necessary for ties in cast concrete, due to excessive vibrations during the "shooting" process.

The choice of shotcrete or cast concrete, for any construction adapted to both materials, can usually be made on a basis of cost. The end results, insofar as deterioration in marine atmospheres are concerned, will be similar. In this connection, it should be borne in mind that the savings, through use of shotcrete, can dwindle or disappear where considerable rebound is incor-

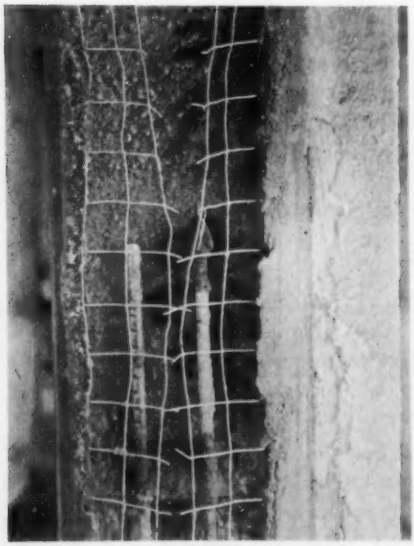

Fig. 5—Close-up during manufacture of solid shotcrete piles. The cement-aggregate mixture was shot onto a half form with wire mesh reinforcement opened up. As the pile approached completion meshes were joined and shotcreting continued

Fig. 6—Prefabricated shotcreted driving tip holds reinforcing cage for solid shotcreted piles. Reinforcing steel from one of these tips is visible in picture at left where pile is being completed

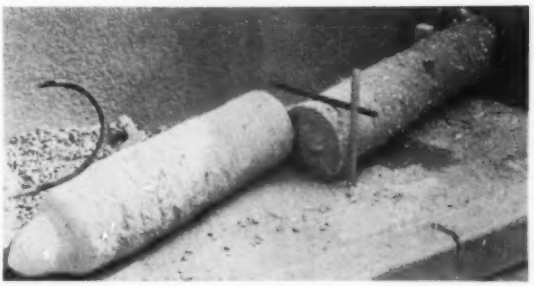

Fig. 7—Cast-in-place pile made of cement slurry mixed with soil as the drilling auger was withdrawn. Test strength in compression was 1655 psi. Soil in which this test pile was made was composed of fine sand and shells from a hydraulic fill

porated in vertical or under surfaces which occasion the removal and replacement of "soft" spots.

Stabilization by grouting

Cement and water, or cement, fine aggregate, and water mixtures may be injected into the ground to stabilize soil and increase its bearing power or to form cut-off walls, cofferdams, and support piling. This can be accomplished in several ways. One such process[27] uses a long rotating drill to loosen the soil to the depth desired, then forces a cement slurry down a hollow auger rod to its tip where it is mixed with the soil as the auger is retracted. If desired, a cage of reinforcing steel may be forced down into the loose grout. The finished product (Fig. 7) hardens to provide a bearing unit or a cut-off wall having any reasonable desired strength, depending on the nature of the soil. It is possible to reach a 58-ft depth and the units may be made in diameters ranging from 12 to 24 in.[2,10,12,30] The Santa Fe Railroad recently used cast-in-place piles for Bridge No. 121-A, near Wilbern, Ill.[8]

Prepacked aggregates

Another adaptation of the same process is the placement of a graded mixture of rock and sand in a form or confined space and pumping a cement slurry through the aggregate by the gradual withdrawal of embedded slurry pipes during the grouting procedure. This process has many applications for marine construction. It may be used both above and below water, for new work (both mass concrete as in the Whittier Narrows Dam), and for reinforced concrete.[12,30,48] It is especially well adapted for marine construction as the binding slurry may be pumped through long pipelines to reach remote locations without danger of plugging even if the contents of the line remain stationary for periods as long as an hour.

Underwater placing

There are other methods of placing concrete mixtures under water for pier construction, such as the bottom dump bucket method used by the Long Beach Harbor Department.[26] Here large rock (100 lb to 6 tons) was placed under water in layers and each layer consolidated by filling the voids with a 6-sack mix concrete (5-7-in. slump) by means of a bottom dump bucket. The final proportion was about 40 percent concrete, 60 percent rock.

Tremie concrete is a concrete or a cement-mortar mixture placed (fresh) under water by means of pumps, buckets, "elephant snouts," etc., where the mixture is usually placed directly on top of the layer previously deposited so that the dilution of the concrete by water is held to a minimum. Concrete so placed is usually stronger than similar mixtures placed in the forms in the air.

ADVANTAGES AND LIMITATIONS OF CONCRETE IN SEA WATER

Concrete is favored as a construction material in marine environments for many reasons. It is composed of readily available materials, which can be used with a minimum of technical skill as compared with other building materials. It is fire resistant, it may be used with complete assurance (within certain usually minor limitations), in the air, underground, and underwater. Construction units may be cast-in-place or precast, using semi-mass production methods and moved to the building site, or assembled there. Its density can be made to vary between limits of 90 and 230 lb per cu ft. Its compressive strength (up to 7500 psi) is ample for most purposes.

Tests made at the Los Angeles Harbor Department testing laboratory indicate that good concrete submerged in sea water increases in strength with the passage of time. Thousands of long-term tests on concrete exposed to sea water show consistent gains in compressive strength.[27] Average compressive strength of 6 x 12-in. cylinders stored 1 year in sea water was 3615 psi; after 20 years exposure in sea water cylinders of the same concrete averaged 5305 psi.

While concrete's tensile and shear strengths are somewhat lower than those of other materials such as wood or steel, they may be considerably improved by modern practices as, for example, prestressing. Being highly resistant to sea water and most marine organisms* when properly designed and placed, it is used by many marine engineers in places where steel or wood are subject to maintenance considerations or comparatively rapid deterioration.

There are several limitations associated with the use of concrete, which are as follows:

1. Some type of form or mold must be employed which usually has to be removed, thereby adding to the cost and delaying subsequent construction.

2. Concrete is slow to attain its working strength. However, construction schedules usually permit sufficient lead time between the completion of concreting and the final opening of the project to enable the concrete to attain its stipulated strength. This limitation is therefore seldom a problem.

*About 35 years ago harbor engineers were alerted to the presence of a new hazard to concrete piles. A clam-like creature, commonly found drilling in shales along the seashore, had taken up underwater residence in some 4-in. "concrete" jackets surrounding wood piles in Los Angeles harbor.[45] This mollusk carried the formidable name of *Pholadidae*. It was later reported in other "concrete" structures. Examination of the "concrete" however, revealed that it was not worthy of the name. In all cases observed by the authors the material could only be classified as a sort of glorified plaster. Coarse aggregate was usually lacking and the cement content was obviously low. The authors have made no observations and have had no reports of this type of attack from *Pholads* or any other marine organisms on good concrete submerged in sea water. It is believed that the so-called rock borers constitute no problem.

3. The required section is frequently large, compared with other materials, which makes it bulky and heavy to use and thus may intensify the foundation bearing problem.

4. For use in marine work where the structures are exposed to the action of sulfates, a low tricalcium aluminate cement is necessary.[11, 43]

5. All reinforcing steel should be embedded at least 3 in.

6. Long precast units such as piles and beams should be handled with equalizing strongbacks or similar arrangements to prevent dangerous overstressing (Fig. 8). As previously mentioned, prestressing tends to alleviate this problem.

7. Concrete is subject to cracking or checking (with commensurate loss of waterproofing qualities) resulting from expansion and contraction, heating and cooling, and/or from wetting and drying.

8. In cold climates alternate freezing and thawing of concrete tends to produce internal stress, which in turn may spall and crumble its surface.

Durability study of piles in service

Although concrete structures in marine environments are generally quite durable, some failures, whose causes were obscure at the time, have been attributed to the use of concrete in sea water. This may be illustrated by an installation of precast, reinforced concrete bearing piles in sea water, which were driven in 1912 as part of the substructure of a 2500-ft concrete wharf in the Los Angeles harbor. The builders took the precaution of employing a competent technical adviser who not only established concrete mix proportions but undertook an extensive laboratory and field research program to develop information on the value of the several types of integral and external concrete waterproofing materials then available. After an elimination test in the laboratory, the best of these were added to different concrete batches when the piles were cast. About 12 years later some of these piles started to crack (longitudinally) beginning at a point just above mid-tide and continuing to the top of the piles (at an elevation of about +14 ft). A number of the pile caps (cast in place at a later date) were likewise cracked. Deterioration progressed rather rapidly and cracking was ultimately accompanied by a marked crumbling of the concrete above the tidal zone and subsequent spalling over the affected area. Research conducted in 1925 disclosed:

1. Concrete below the water line was perfectly sound and durable.

2. Chemical reactions had taken place within the tidal range (assumed to be a reaction between magnesium sulfate in the sea water and lime compounds in the cement[4, 21] which progressively produced a larger molecular structure than the original one, within the interstices of the concrete. The expansion forces of the new chemical formation exceeded restraining ability of the concrete, resulting in fissures allowing sea water to come in direct contact with the reinforcing steel.

3. A secondary chemical reaction took place with the formation of a hydroxide of iron around the reinforcing steel thus causing a longitudinal cracking of the pile over the reinforcing bars.

Corrosion of reinforcing steel

Today, it is the conviction of the writers that, (except in the case of C_3A, discussed later), chemical reactions[4] between concrete and sea water are of academic interest only and can be ruled out[37] as a major deterioration cause, and that the problem of the corrosion of the reinforcing steel is actually the

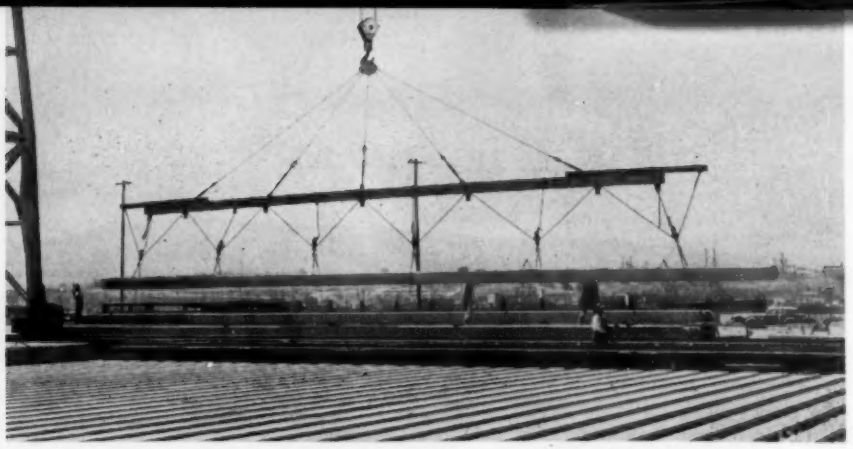

Fig. 8—A multiple-point suspension strongback used to alleviate bending stresses in the handling of precast reinforced concrete piles

primary issue. This explains why failures, often attributed to chemical reactions, have occurred in reinforced concrete while mass concrete structures under similar exposure have remained sound.[28,49]

Recent studies of Tremper, Beaton, and Stratfull,[41,46] in northern California, show that sea salts, when absorbed by reinforced concrete, give rise to corrosion cells wherein anodic and cathodic areas are established at distances which may be several feet apart. Corrosion products formed on the anodic areas of the reinforcing steel develop in sufficient quantities to rupture the surrounding concrete. Although the study was not specifically directed toward precautions in constructing new concrete, their data support the premise that the activity of such corrosion cells would be greatly reduced by the use of low water-cement ratio concrete containing well graded aggregates, and by increasing the depth of cover over the reinforcing steel.

Tremper *et al.*[41,46] observed random failures of concrete with adjacent areas remaining sound. It seems that the cause must be permeable concrete cover in the deteriorated regions, with concrete impermeable in the sound regions. Why the heterogeneity in the concrete cover existed was not included in this particular study. However, the authors are convinced that it is possible to cast uniform concrete, given adequate specifications and inspection.

W. J. Copenhagen, South African Council for Scientific and Industrial Research, Capetown, while working independently along the same lines as Tremper, Beaton, and Stratfull, has concluded some interesting work[13] of a somewhat different nature which may help to explain more fully the phenomenon of anodic and cathodic areas on reinforced marine concrete structures. He concludes that the corrosion of steel embedded in concrete is curtailed by a protective film of gamma ferric oxide in the presence of the inhibiting effect of high pH in areas where calcium hydroxide is present. When calcium hydroxide is converted to calcium carbonate, (from diffusion of carbon dioxide in atmosphere) the pH is lowered and corrosion takes place as the protective film is destroyed.

Furthermore, it is stated that the presence of salt (in amounts greater than 0.1 percent) from sea water deposits augments the specific electrical conductivity of the concrete and simultaneously destroys the protective film on the steel, initiating corrosion through normal electrochemical processes.

A procedure is given for determining the conversion of calcium carbonate inward from the surface of the concrete by means of a pH indicator solution (thymolphthalein) which produces a blue coloration in the pH ranges of 9.3 to 12.5. Calcium hydroxide lies within this blue range while calcium carbonate does not.

Summarizing the results of two independent investigations, Tremper *et al.* and Copenhagen, we find that:

1. Permeability of concrete in any given job may be far from constant.
2. The more permeable areas permit carbon dioxide in the air to convert calcium hydroxide (formed in the concrete in the capillary zone) to calcium carbonate.
3. Calcium hydroxide, having a higher pH value than the calcium carbonate, affords greater protection to the reinforcing steel.
4. Variations in pH over the steel surface, especially in the presence of salt, give rise to anodic and cathodic areas.
5. Corrosion of the steel follows in normal sequence.

All of these facts point to the possibility of making concrete which will last indefinitely in a marine environment. Many authorities hold that there is no attack within the concrete unless sea water penetrates the cover over the reinforcing steel and can react with the mass.[21]

To prevent deterioration arising from corrosion of reinforcement, the writers recommend that 3 in. of good, homogeneous concrete cover be used over reinforcing steel to insure permanence. There have been cases where as little as 1½ in. has sufficed. But there have also been many failures of structures of identical design, indicating the need for more protection, or an adequate margin of safety against the ever threatening possibility of anything less than the most skilled workmanship.

Physical reaction

A physical reaction of concrete in sea water may occur if the concrete is poorly graded with respect to particle sizing of the aggregates, thus making it porous, and susceptible to water absorption. Subsequent evaporation of sea water from the surface interstices of porous concrete in the tidal zone, may cause crystal growths (magnesium sulfate, calcium sulfate, sodium chloride, etc.) which can exert considerable swelling force and eventually progressively crumble the surface of the concrete.[47]

Sulfate attack

Although the authors discount the probability of chemical reactions as being of prime importance in the deterioration of good concrete in sea water there is ample evidence for one exception* when critical quantities of tricalcium aluminate (C_3A) occur in the cement used for concrete.

*See References 5, 6, 11, 19, 22, 25, 31, 33, 39, 40, 44, and 50.

Available information indicates that calcium sulfoaluminate is present in all concrete as a result of interaction of C_3A and gypsum. Petrographers state that in sound concrete the sulfoaluminate crystals are uniformly distributed. It is harmless when it is the natural reaction during the setting of concrete. The capacity of the concrete to form *additional* calcium sulfoaluminate with sulfates from sea water depends upon the amount of residual C_3A after sulfate from the gypsum is depleted. The stresses formed from this additional growth can be sufficient to rupture the surface. When concrete is stressed from chemical causes, calcium sulfoaluminate needles are found in concentrated crystal masses instead of being uniformly distributed as in sound concrete.

To minimize this crystal formation, a Type V cement having 5 percent maximum C_3A is usually specified for marine construction. When Type V is economically infeasible, a Type II cement is a good second choice. In some locations throughout the country, due to a fortunate combination of raw materials, a nominal Type I cement having some or all of the desirable properties of a Type V or Type II cement is available. In such special cases, the more expensive types may not be warranted.

The sulfates of sea water may also combine with calcium hydroxide in the concrete to form calcium sulfate,[3] but since this salt is relatively soluble it is probably removed almost as fast as it is formed. On the other hand, magnesium salts precipitated in the concrete are probably partially responsible for the increase in volume[21] as is the case with the formation of calcium sulfoaluminate crystals in sea water.[5]

Reactive aggregates

Another chemical reaction which must be considered in using concrete in any environment is that encountered when reactive aggregates are used (see References 1, 7, 24, 36, 37, and 38). Certain types of cement which have relatively high alkali contents (in excess of 0.6 percent total Na_2O and K_2O*) tend to react with aggregates containing hydrated silica, such as chert, chalcedony, and certain types of shale, to form complex alkali-like silicates which may rupture and crack concrete. The solution is to avoid the use of reactive aggregates. If they must be used, a low-alkali cement, together with a pozzolanic admixture† such as calcined Monterey shale, should be employed. This will tend to alleviate the problem. Undesirable effects sometimes result when pozzolans are employed, such as low early strength and somewhat greater volume changes in the concrete during wetting and drying. Many investigators are convinced that the use of low-alkali cements can offset alkali-aggregate reactions without the need for pozzolans.

Air entrainment

Concrete subjected to freezing action has a tendency to crumble and spall. The deteriorated concrete resembles that which has been disintegrated by

*Calculated as Na_2O (equivalent $Na_2O + 0.658\ K_2O$).
†See References 14, 15, 16, 18, 29, 37, 42 and 47.

sea water, and the cause may be generally the same—the expansion of growing crystals. In this case they are ice crystals. A partial remedy for this situation is to entrain air cells in the concrete as it is being made by the addition of various chemicals (see References 11, 17, 20, 23, 34, 43, and 51). An alternative would be to employ precast, asphalt impregnated concrete units. Air-entraining agents are also often desirable to improve the workability of the concrete, and to effect a substantial reduction in its original moisture content. Popular among these air-entraining agents is a compound known as a neutralized Vinsol resin. This is added to the concrete in the mixer as a prepared liquid to provide a suitable range of air entrainment, (depending on environment, size of aggregate, etc.). Three to six percent of entrained air is usually specified when needed for marine structures, where the coarse aggregate size ranges from 1½ to 3 in.

CONCLUSIONS

It is possible to produce concrete for marine structures which will endure without failure at least 50 years with no sign of deterioration in that interval. The following precautions should be observed for concrete units which are to be exposed to the action of sea water.

1. Proper mix proportions using the optimum cement content and competent workmanship are necessary to provide a dense, impervious, relatively nonabsorbent concrete.
2. Embed all reinforcing steel so that it has a minimum cover of 3 in. of good concrete.
3. Use a minimum cement content of 6½ sacks per cu yd, with a maximum of 7½ sacks.
4. Use nonreactive aggregates, graded to obtain maximum density. If reactive aggregates must be employed, use a low-alkali cement and/or compensate with pozzolans.
5. Use sufficient water in the concrete to produce a plastic, workable mix, having low water-cement ratio, usually not to exceed a total of 6 gal. per sack of cement.
6. Use Type V cement, having 5 percent maximum C_3A. If this type is not economically feasible, a Type II cement may be substituted.
7. In the case of marine concrete which is to be exposed to freezing and thawing action, it is important to use some type of air-entraining agent.
8. Thoroughly cure the concrete by the best means possible.
9. Particular care should be taken not to damage precast units such as long piling, by improper handling. They should be lifted with equalized, multiple point suspension slings, or similar devices, wherever necessary. Prestressed piles are less susceptible to handling and tensile stresses than are ordinary reinforced piles.

With the exception of the items relating to potentially adverse chemical reactions, all of the above precautions indicate one thing, i.e., the necessity to waterproof the concrete as thoroughly as possible, using the most feasible and economic means available. The term "waterproof" is understood to mean not only prohibiting the free passage of moisture, but reducing its absorption to a stipulated minimum.

This paper, for all practical purposes, is intended merely as an expanded outline concerning the use of concrete in a marine environment. It was written with the hope of stimulating further interest in more specialized papers on the same subject in the future.

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REFERENCES

1. "Alkalies in Cement and Their Effect on Aggregates and Concretes," *Proceedings*, U. S. Bureau of Reclamation Conference, Feb. 14-15, 1941.

2. *Proceedings*, ASCE, Feb. 1954.

3. Atwood, William G., and Johnson, A. A., "The Disintegration of Cement in Sea Water," *Transactions*, ASCE, V. 87, 1924, pp. 204-230; discussion pp. 231-275.

4. Bates, P. H., Phillips, A. J., and Wig, R. J., *Technologic Paper* No. 12, National Bureau of Standards, Nov. 1, 1912.

5. Bogue, R. H., "Blast Furnace Slag in Concrete," ACI JOURNAL, Oct. 1955, *Proc.* V. 52, pp. 227-230.

6. Bogue, R. H., Lerch, William, and Taylor, C. W., "Influence of Composition on Volume Consistency and Salt Resistance to Portland Cement Pastes," *Industrial and Engineering Chemistry*, V. 36, 1934, pp. 1049-1060.

7. Blanks, R. F., "Modern Concepts Applied to Concrete Aggregates," *Proceedings*, ASCE, V. 75, 1949, pp. 441-468; *Transactions*, ASCE, V. 115, 1950, pp. 403-31.

8. "Bridge on Santa Fe Shows Versatility of Concrete Construction," *Concrete for Railways*, V. 13, No. 1.

9. Briggs, L. J., (acting director, National Bureau of Standards) unpublished letter dated Aug. 10, 1932.

10. Liver, Norman L., Hardoff, Erwin C., and King, John C., "Mixed-in-Place Piles Form Jetty to Control Beach Erosion," *Civil Engineering*, Mar. 1954, p. 56.

11. Cook, Herbert K., "Experimental Exposure of Concrete to Natural Weathering in Marine Locations," *Proceedings*, ASTM, V. 52, 1952, pp. 1179, 1180.

12. Cook, Herbert K., "Two Test Structures of Prepakt Concrete Completed at Vicksburg Experiment Station," *Civil Engineering*, Mar. 1950, p. 40.

13. Copenhagen, W. J., "Preliminary Investigation of the Atmospheric Corrosion of Reinforcing Steel in Concrete Structures in Marine Environments," *African Builder*, Dec. 1953 (and private communications).

14. Davis, R. E., "Use of Pozzolans in Concrete," ACI JOURNAL, Jan. 1950, *Proc.* V. 46, pp. 377-384.

15. Davis, R. E., Hanna, W. C., and Brown, E. H., "Strength, Volume Change, and Sulfate Resistance of Mortars Containing Portland-Pozzolan Cements," Symposium on Use of Pozzolanic Materials in Mortars and Concretes, *ASTM Special Technical Publication* No. 99, 1949, p. 131.

16. Davis, R. E., and Polivka, M., discussion of "The Significance of Tests for the Sulfate Resistance of Concrete," *Proceedings*, ASTM, V. 53, 1953, p. 1015.

17. Gonnerman, H. F., "Tests of Concretes Containing Air-Entraining Portland Cements or Air-Entraining Materials Added to Batch at Mixer," ACI JOURNAL, June 1944, *Proc.* V. 40, pp. 477-507; discussion pp. 508-1 to 508-6.

18. Hanna, W. C., discussion of "The Significance of Tests for the Sulfate Resistance of Concrete," *Proceedings*, ASTM, V. 53, 1953, p. 1012-1013.

19. Higginson, E. G., and Clantz, O. J., "The Significance of Tests for Sulfate Resistance of Concrete," *Proceedings*, ASTM, V. 53, 1953, p. 1002.

20. Jackson, Frank H., and Timms, Albert C., "Evaluation of Air-Entraining Admixtures for Concrete," *Public Roads*, V. 27, 1954, pp. 259-267.

21. Lea, F. M., and Desch, C. H., *The Chemistry of Cement and Concrete*, Edward Arnold & Co., London, 1st edition, 1935, p. 365.

22. LeChatelier, H., "Decomposition of Cements in Sea Water," *Official Reports*, International Association for Testing Materials, V. 31e, 1906, p. 16.

23. Lerch, William, "Basic Principles of Air-Entrained Concrete," Portland Cement Association, 1953, p. 36.

24. Lerch, William, "Significance of Tests for Chemical Reactions," *Proceedings*, ASTM, V. 53, 1953, p. 978.

25. Lerch, W., Ashton, F. W., and Bogue, R. H., "The Sulfoaluminates of Calcium," *Research Paper* No. 54, National Bureau of Standards, 1929.

26. Long Beach, Calif., Harbor Department records.

27. Los Angeles Harbor Department testing laboratory pertinent records.

28. Mather, Bryant, "Factors Affecting Durability of Concrete in Coastal Structures," *Technical Memorandum* No. 96, Beach Erosion Board, U. S. Army, Corps of Engineers, June, 1957.

29. Meissner, H. S., "Pozzolans Used in Mass Concrete," Symposium on Use of Pozzolanic Materials in Mortars and Concretes, *ASTM Special Technical Publication* No. 99, 1949, p. 16.

30. "Memorandum for the Chief, Military Construction Division on the Subject of Prepakt Concrete," War Department, Corps of Engineers, Office of the District Engineer, New York District, New York, Mar. 21, 1947.

31. Miller, Dalton G., "Sulfate Resistant Cement—Primary Requirement for Sulfate Resistant Concrete Pipe," ACI JOURNAL, Nov. 1952, *Proc.* V. 49, pp. 217-224.

32. Nicholson, G. F., "Protecting Reinforced Concrete Against Deterioration by Sea Water," *Civil Engineering*, Nov. 1931, p. 1243.

33. O'Hara, J. M., "Action of Sea Water on Portland Cement," *Engineering News-Record*, V. 61, 1910, p. 677.

34. Powers, T. C., "Void Spacing as a Basis for Producing Air-Entrained Concrete," ACI JOURNAL, May 1954, *Proc.* V. 50, pp. 741-760; discussion pp. 760-1 to 760-15, Part 2, Dec. 1954.

35. "Recommended Practice for Measuring, Mixing and Placing Concrete (ACI 614-42)," reported by ACI Committee 614, ACI JOURNAL, June 1945, *Proc.* V. 41, pp. 625-650.

36. Stanton, T. E., *et al.*, "California Experience with the Expansion of Concrete Through Reaction Between Cement and Aggregate," ACI JOURNAL, Jan. 1942, *Proc.* V. 38, pp. 209-236.

37. Stanton, T. E., "Durability of Concrete Exposed to Sea Water and Alkali Soils—California Experience," ACI JOURNAL, May 1948, *Proc.* V. 44, pp. 834, 839.

38. Stanton, T. E., "Expansion of Concrete Through Reaction Between Cement and Aggregate," *Transactions*, ASCE, V. 107, Paper No. 2129, 1942, p. 54.

39. Stanton, T. E., "Tests Show Resistance to Sea Water of California Cements," *Shore and Beach*, V. 6, 1938, pp. 34-36.

40. Stanton, T. E., and Meder, L. C., "Resistance of Cement to Attack by Sea Water and by Alkali Soils," ACI JOURNAL, Mar.-Apr. 1938, *Proc.* V. 34, pp. 433-464.

41. Stratfull, R. F., "The Corrosion of Steel in a Reinforced Concrete Bridge," *Corrosion*, V. 13, No. 3, Mar. 1957, p. 43.

42. Symposium on Use of Pozzolanic Materials in Mortars and Concretes, *ASTM Special Technical Publication*, No. 99, 1950.

43. "Ten-Year Report on the Long Time Study of Cement Performance in Concrete," reported by Advisory Committee, Long-Time Study, ACI JOURNAL, Mar. 1953, *Proc.* V. 49, pp. 601-616.

44. "The Conditions for the Formation and Existence of Calcium Aluminosulfate," *Izvestiya Akademii Nauk S.S.S.R., Otdelenie Tekhnicheskikh Nauk* (Moscow), 1937, pp. 125-148.

45. "The Occurrence of Rock Boring Mollusks in Concrete," *Marine Borers and Their Relation to Marine Construction on the Pacific Coast*, final report of the San Francisco Bay Marine Piling Committee, 1927, Chapter XIX, p. 301.

46. Tremper, Bailey, Beaton, John L., and Stratfull, R. F., "Causes and Repair of Deterioration to a California Bridge due to Corrosion of Reinforcing Steel in a Marine Environment. Part 2—Fundamental Factors Causing Corrosion," *Bulletin* No. 182, Highway Research Board, 1957.

47. Tuthill, Lewis H., "Resistance of Cement to the Corrosive Action of Sodium Sulfate Solutions," ACI JOURNAL, Nov.-Dec. 1936, *Proc.* V. 33, pp. 97, 104-105.

48. "Investigation of the Suitability of Prepakt Concrete for Mass and Reinforced Concrete Structures," U. S. Army Corps of Engineers, Waterways Experiment Station, *Technical Memorandum* No. 6-330, Oct. 1951.

49. Wakeman, C. M., "27-Year-Old Concrete in Sea Water," unpublished Los Angeles Harbor Department report (Lab. No. 2166), July 15, 1932.

50. Watson, W., and Craddock, Q. L., "Deterioration of Concrete Structures in Alkaline and in Sea Water," *Cement and Cement Manufacture* (London), V. 8, 1935, pp. 130-141.

51. Woods, Hubert, "Observations on the Resistance of Concrete to Freezing and Thawing," ACI JOURNAL, Dec. 1954, *Proc.* V. 51, p. 347; discussion, pp. 352-1 to 352-10, Part 2, Dec. 1955.

Discussion of this paper should reach ACI headquarters in triplicate by July 1, 1958, for publication in the Part 2, December 1958 JOURNAL.

Title No. 54-47

Some Physical Properties of Concrete at High Temperatures*

By ROBERT PHILLEO†

SYNOPSIS

Experimental techniques are described and data are presented on the thermal expansion, density, and dynamic modulus of elasticity of concrete in the range 75-1500 F. Such information is necessary to evaluate stresses due to non-uniform heating which could result from a building fire or jet aircraft blast. The results indicate that weight loss due to loss of water is substantially complete at 800 F. At higher temperatures changes in weight are determined by the chemical nature of the aggregates. The coefficient of expansion increases above 800 F since expansion is no longer inhibited by drying shrinkage. At 1400 F the modulus of elasticity is reduced to less than half its value at 75 F, the exact reduction depending on the extent to which hydration had progressed at the time of exposure.

Although most concrete is subjected to a range of temperature no more severe than that imposed by weather, there are important cases in which it is exposed to much higher temperatures. Examples include building fires, jet aircraft blast, and some industrial applications where concrete is close to furnaces and reactors. In such applications the effect of the high temperature on certain physical properties of the concrete may determine whether it maintains its structural integrity. In connection with an investigation of the fire resistance of concrete conducted at the Portland Cement Association Research and Development Laboratories, determinations of linear expansion, weight loss, and dynamic modulus of elasticity were made under several conditions in the range 75-1500 F. The thermal expansion and modulus of elasticity data are prerequisite to computing thermoelastic stresses resulting from non-uniform heating. No such information has been reported previously.‡ Measurements were made as equilibrium was attained at several temperatures during the first heating cycle. Hence the results are applicable to concrete which is maintained for long periods of time at high temperature and to concrete exposed to a fire, but not necessarily to concrete subjected to temperature cycling.

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†Member American Concrete Institute, Research Engineer, Fire Research Section, Portland Cement Association, Chicago, Ill.

‡Since the original preparation of this paper, "Variation of Mortar and Concrete Properties with Temperature," by J. C. Saemann and G. W. Washa, appeared in the ACI JOURNAL, Nov. 1957, *Proc.* V. 54, pp. 385-396. Study therein reported covered temperatures between − 70 and 450 F.—EDITOR

SCOPE OF TESTS

Complete tests were made on a group of specimens of three water-cement ratios: 0.4, 0.6, and 0.8 by weight, containing a blend of four Type I cements and sand and gravel (maximum size ¾ in.) from Elgin, Ill. The range in cement contents was 4.4-7.8 sacks per cu yd. The group included both specimens continuously moist cured and specimens moist cured for 3 days and air dried at 50 percent relative humidity until tested. Test ages were 28 and 90 days. A few additional tests were made on cement paste specimens and on concretes containing expanded shale aggregate. Reported results are based on one or three specimens as indicated in the tables.

APPARATUS

In all tests specimens were supported in a firebrick frame. The open sides of the frame were enclosed by a pair of electric furnaces built in the laboratory. Each contained eight ½ in. diameter carborundum "Globars" in a firebrick housing with a 4 x 10-in. opening on one side.

Fig. 1 shows the details of the expansion and weight change measurement apparatus. The lower specimen is undergoing expansion measurements while the upper specimen is being weighed. For the thermal expansion measurements, vertical grooves were sawed across the middle of each end face of a 2 x 2¾ x 6-in. specimen. Number 30 gage Chromax wires were draped over the edges of the specimen through the grooves. The wire extended through holes in the bottom of the frames, and about 1 ft below the frames weights were attached to maintain

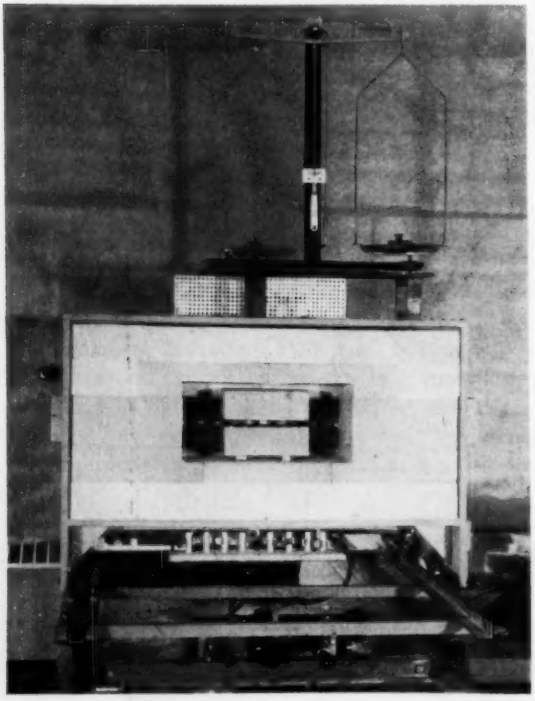

Fig. 1—Length change and weight loss apparatus

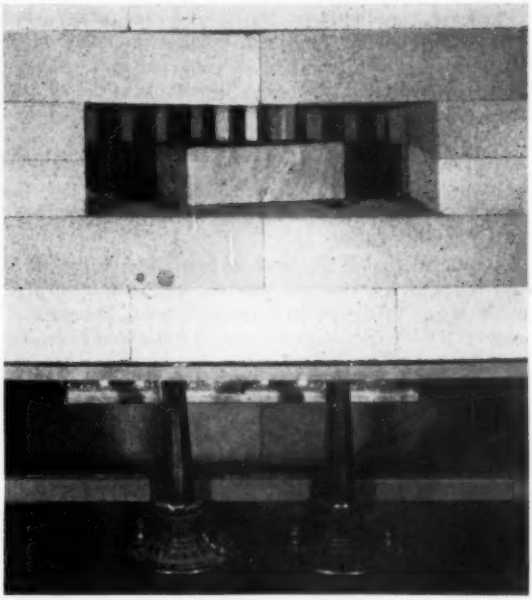

Fig. 2—Resonant frequency apparatus

the wires vertical. The weights and lower portion of the wires were immersed in an oil bath to eliminate swaying and vibration of the wires. Horizontal movements of the wires, hence the changes in length of the specimen, were measured with a pair of micrometer microscopes having a readability of 1 micron. Thus increments of length change equal to 7×10^{-6} of the specimen length could be detected.

For the weight loss measurements, one end of a 2x2¾x6-in. specimen was suspended from a wire attached to the ceiling of the frame opening while the other end was suspended from a wire which extended up through a small hole in the top of the frame to one arm of a balance mounted atop the frame. One half of the specimen was weighed continuously by maintaining the balance level. Assuming that weight was being lost symmetrically from the specimen, its weight was known at all times.

The modulus of elasticity of 1½x2x6-in. specimens was obtained by determining the resonant frequency of the specimens in flexural vibration inside the furnace. This was accomplished by supporting the specimens at opposite corners on the small ends of truncated steel cones which served as the driving and pickup units as seen in Fig. 2. The cones, 11 in. high with end diameters of ¼ and 1¼ in., were attached at the larger ends to University model MA-25 "tweeter" loud-speakers. The tips of the cones extended ⅛ in. into the hot zone through holes in the bottom of the furnace specimen frame. The technique of driving the specimens through air is an adaptation of a method reported by Spinner.* The rest of the equipment was that normally used for the sonic test and consisted of audio-oscillator, amplifier, vacuum-tube voltmeter, cathode-ray oscilloscope, and frequency meter.

Use of the loud-speaker as a pickup eliminated high temperature problems that would have been present if a crystal pickup had been used. Although the tips of the cones reached temperatures above 1500 F, the temperature at the base of the cones never became high enough to impair speaker operation. By supporting the specimens at opposite corners it was possible to induce and detect flexural vibrations in two directions as well as torsional vibrations. In

*Spinner, Sam, "Elastic Moduli of Glasses by a Dynamic Method," *Journal*, American Ceramic Society, V. 37, No. 5, pp. 229-234.

addition the two air columns, which served as the acoustical coupling medium, introduced a number of incidental resonant frequencies. The concrete frequencies could be separated from the air frequencies by virtue of the fact that with rising temperatures concrete frequencies decrease whereas air frequencies increase. As the temperature was being changed, frequency changes of only a few cycles could be tracked by observing phase shifts on the oscilloscope. Another means of identification was provided by the constant ratio between the two flexural frequencies and the nearly constant ratios between torsional and flexural frequencies. In addition to the frequencies, the weight of the specimen at each temperature must be known in order to compute the modulus of elasticity. This information was available from the weight tests described above.

In all tests the air temperature was raised in discrete steps of about 200 F at a time; each new temperature was held constant until equilibrium was attained both in specimen temperature and in the physical property being measured. This usually required a period not exceeding 3 hr, although on occasion static conditions were maintained as long as 17 hr with no appreciable change in results. A temperature record was obtained on a Brown recorder from thermocouples in the air of the furnace and either in the specimen or between the specimen and its support.

TEST RESULTS

Thermal expansion

The length-change measurements plotted against temperature in the gravel concretes consistently produced data which could be represented by two straight lines with a short transition section between them. The slope of the

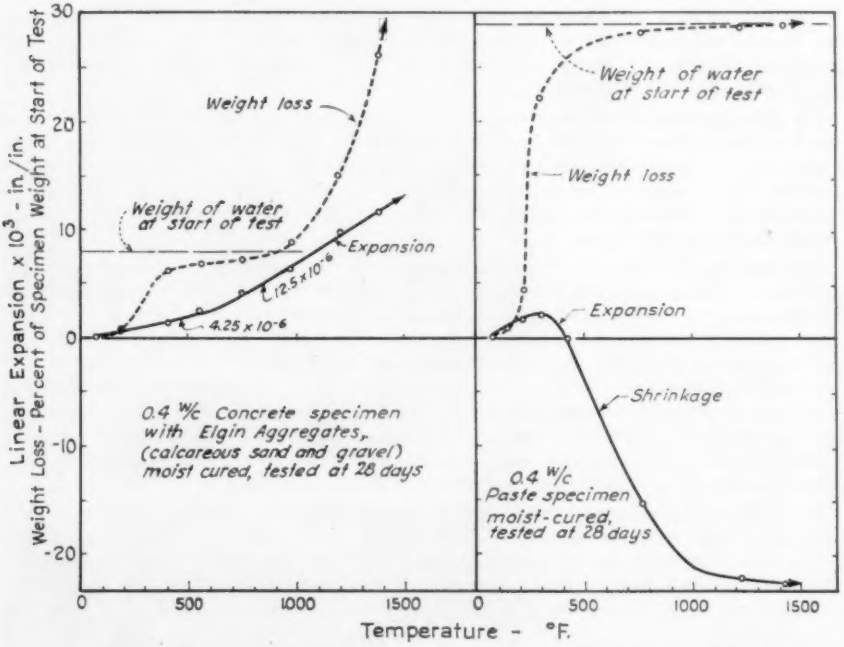

Fig. 3—Expansion and weight loss of gravel concrete and cement paste

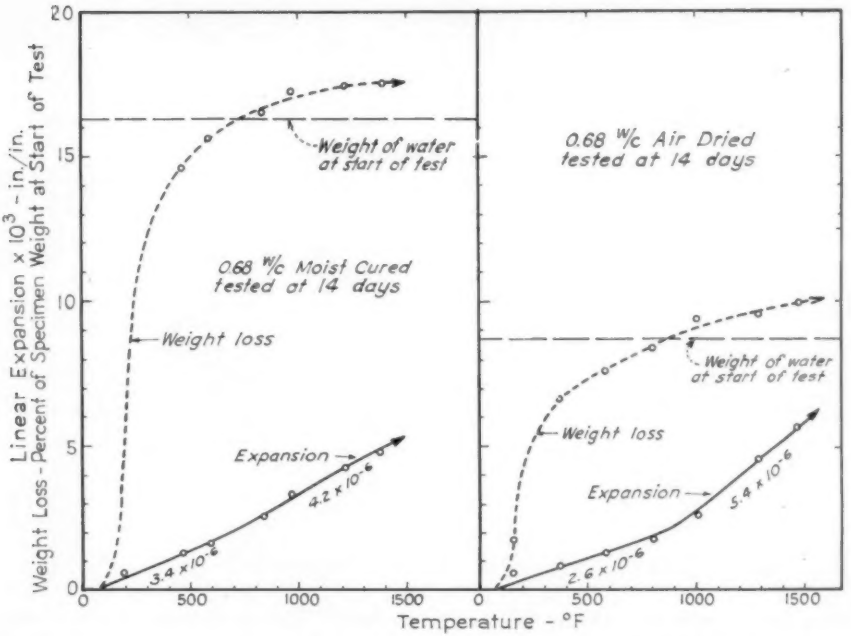
Linear Expansion x 10^3 - in./in.
Weight Loss - Percent of Specimen Weight at Start of Test
0.68 W/c Moist Cured tested at 14 days
Weight of water at start of test
Weight loss
Expansion
3.4 x 10^{-6}
4.2 x 10^{-6}
0.68 W/c Air Dried tested at 14 days
Weight of water at start of test
Weight loss
Expansion
2.6 x 10^{-6}
5.4 x 10^{-6}
Temperature - °F

Fig. 4—Expansion and weight loss of concretes with expanded shale aggregate

first is the linear coefficient of expansion below about 500 F, and the slope of the second the coefficient above 800 F. Fig. 3 shows a typical set of results for a single gravel concrete specimen and for cement paste alone. On the same drawing weight losses are shown. The increase in coefficient of expansion with increasing temperature is also present in expanded shale concrete but to a lesser degree. Results for a moist cured and air dried expanded shale concrete are shown in Fig. 4. Values of the coefficients for the various materials, water-cement ratios, curing conditions, and ages are shown in Table 1. Above 800 F most of the gravel concretes have about the same coefficients. All the results were obtained for the first heating cycle with each specimen and are not reversible at high temperatures. At temperatures above 1000 F specimens cool with a lower coefficient, indicating that some permanent dilation has occurred. As mentioned before, however, these values are believed to be applicable to concretes exposed to fire and to concretes permanently exposed to high temperature.

Weight loss

It has been observed* that exposure of concrete to temperature around 1000 F for a prolonged period will reduce the weight essentially to that of the original dry ingredients. The dashed curves in Fig. 3 and 4 verify this and

*Blackman, James S., "Method of Estimating Water Content of Concrete at the Time of Hardening," ACI JOURNAL, Mar. 1954, *Proc.* V. 50, pp. 533-544.

TABLE 1—LINEAR COEFFICIENTS OF THERMAL EXPANSION

Water-cement ratio, by weight	Cement content, sacks per cu yd	Linear coefficient of thermal expansion at 28 days, per deg F		Linear coefficient of thermal expansion at 90 days, per deg F	
		Below 500 F	Above 800 F	Below 500 F	Above 800 F
Concrete containing Elgin sand and gravel					
Moist cured, average of three specimens per test					
0.4	7.82	4.2×10^{-6}	11.3×10^{-6}	3.6×10^{-6}	6.2×10^{-6}
0.6	5.52	7.1×10^{-6}	11.4×10^{-6}	4.7×10^{-6}	12.5×10^{-6}
0.8	4.43	6.1×10^{-6}	11.7×10^{-6}	9.3×10^{-6}	18.2×10^{-6}
Air dried, one specimen per test					
0.4	7.82	4.3×10^{-6}	10.5×10^{-6}	6.8×10^{-6}	11.5×10^{-6}
0.6	5.52	4.3×10^{-6}	11.7×10^{-6}	4.9×10^{-6}	11.2×10^{-6}
0.8	4.43	5.3×10^{-6}	11.5×10^{-6}	6.5×10^{-6}	12.0×10^{-6}
Concrete containing expanded shale aggregate, one specimen per test					
Water-cement ratio	Cement content, sacks per cu yd	Linear coefficient of thermal expansion at 14 days, per deg F		Linear coefficient of thermal expansion at 21 days, per deg F	
		Below 500 F	Above 800 F	Below 500 F	Above 800 F
Moist cured					
0.68	6.36	3.4×10^{-6}	4.2		
Air dried					
0.68	6.36	2.6×10^{-6}	5.4	2.8×10^{-6}	4.9

NOTE: Air-dried specimens were moist cured 3 days, then dried at 50 percent relative humidity.

also show complete weight-temperature data. The horizontal broken line shows the total water, including combined water, present at the start of the test. In the concretes containing expanded shale the weight loss exceeded the weight of water thought to be present. Either moisture in the aggregate at the time of mixing was incorrectly determined or some constituent of the shale volatilized.

The Elgin aggregates are largely calcareous and at high temperatures carbon dioxide is driven off with a large attendant loss of weight. This does not occur, however, until the temperatures are above the temperature at which substantially all the water has been removed.

These weight effects also offer an explanation of the expansion curves. The right half of Fig. 3 shows a pronounced shrinkage of the paste which is much greater than its own thermal expansion during most of the drying period (250-1000 F). This shrinkage undoubtedly tends to counteract the thermal expansion of concrete at the lower temperatures. The transition to the higher expansion coefficient occurs after the drying is nearly complete. The existence of a more definite second coefficient in the gravel concrete than in the expanded shale suggests that decarbonation affects the expansion characteristics of the gravel. This is also indicated by the fact that the value of 11×10^{-6} per deg F common to the gravel concretes at high temperatures is higher than values determined on Elgin aggregates alone at normal temperatures.

Modulus of elasticity

The modulus of elasticity underwent drastic reductions in all test specimens as the temperature was increased. As in the case of thermal expansion, speci-

TABLE 2—DYNAMIC MODULUS OF ELASTICITY AT HIGH TEMPERATURES

Concrete containing Elgin sand and gravel, one specimen per test

	0.4 *W/C*		0.6 *W/C*		0.8 *W/C*	
	Moist cured	Air dried	Moist cured	Air dried	Moist cured	Air dried
28-day results						
$E \times 10^{-6}$ at 75 F	6.14	5.14	5.31	4.09	4.53	3.20
	E, percent of modulus of elasticity at 75 F					
Temperature, F						
75	100	100	100	100	100	100
200	87	89	89	86	97	88
400	73	75	72	69	68	65
600	64	63	60	59	54	56
800	56	55	52	53	43	52
1000	48	49	43	49	36	47
1200	41	45	36	45	29	37
1400	36	43	29	40	22	31
$E \times 10^{-6}$ at 1400 F	2.19	2.20	1.57	1.65	1.01	0.99
90-day results						
$E \times 10^{-6}$ at 75 F	7.34	5.48	6.34	4.55	5.50	3.50
	E, percent of modulus of elasticity at 75 F					
Temperature, F						
75	100	100	100	100	100	100
200	88	83	87	89	84	78
400	73	68	66	74	64	59
600	63	61	53	65	51	50
800	54	55	49	60	46	47
1000	45	50	43	56	42	42
1200	37	45	36	43	29	32
1400	31	39	28	37	21	26
$E \times 10^{-6}$ at 1400 F	2.30	2.15	1.77	1.69	1.18	0.91

NOTE: Air dried specimens were moist cured 3 days, then dried at 50 percent relative humidity.

mens tended after dehydration to have the same low modulus regardless of age or curing conditions. There was, however, an effect of original water-cement ratio. The lower the water-cement ratio, the higher the modulus of elasticity after dehydration. The results are summarized in Table 2. It may be seen that the further hydration has progressed, the greater the reduction in the modulus as a result of heating. Thus the greatest reduction occurred in a concrete with 0.8 water-cement ratio continuously moist cured for 3 months, and at each water-cement ratio the moist cured specimens underwent greater reductions than the companion dried specimens.

Since both flexural and torsional frequencies were determined, it was possible to compute Poisson's ratio. There was a general tendency for Poisson's ratio to decrease as the temperature rose, although the results were erratic. The calculation of Poisson's ratio is very sensitive to errors in determining the resonant frequencies. A 1 percent error in frequency may produce as much as a 20 percent error in Poisson's ratio.

SUMMARY

The performance of concrete exposed to high temperature is influenced by the effect of the temperature on several important physical properties. Those investigated in this study were thermal expansion, density, and modulus of elasticity. The results can be explained principally on the basis of the dehydration of cement paste. Decarbonation of calcareous aggregates also

plays a part particularly in regard to weight loss. The coefficient of thermal expansion of concrete is numerically greater above 800 F than below, since below that temperature it is affected by drying shrinkage of the paste. On heating to 1400 F the modulus of elasticity of the concretes tested was reduced to less than half the value at 75 F. The exact reduction depended on age, mix proportions, and curing conditions. For a given water-cement ratio the modulus of elasticity after dehydration was independent of age or curing conditions.

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Title No. 54-48

Flexural Cracks in Reinforced Concrete Beams*

By MICHAEL CHI† and ARTHUR F. KIRSTEIN‡

SYNOPSIS

A new concept is introduced into the problem of crack formation in reinforced concrete beams subjected to pure flexure, along with simplified semi-empirical equations for the determination of the average minimum spacing and the average width of cracks in the concrete. Historical background given leads to development of the analysis and the assumptions incorporated in it. To verify the analysis, crack data from 16 test specimens are presented and used to augment the data from previous investigations.

INTRODUCTION

It is well known that the design load of a reinforced concrete flexural member will produce cracks in the tensile zone of the concrete. While hairline cracks are considered inevitable, cracks of appreciable width are intolerable because they may be a source of danger due to the possible corrosion of the reinforcing steel. Furthermore, cracks wide enough to be visible tend to destroy the aesthetic value of the structure.

In order to limit the width of tensile cracks the usual design practice is to "over-reinforce," and current building codes limit the allowable stress to about 20,000 psi regardless of the steel strength. Even though steel manufacturers have long been able to produce steel with ductility equal to and strengths greater than those of the steels used in establishing the 20,000-psi limit, it is felt by some that any increase of the allowable stress would cause excessive cracking and deflection. With this limitation the higher strength steels have not been used.

Since it is recognized that crack width increases in a nearly linear relationship with the stress in the steel, restrictions imposed by the building codes are not without basis. However, it has also been established that many other factors also influence the crack width. Conceivably, with proper control of these factors crack widths can be kept within a tolerable margin even with steel stresses much higher than 20,000 psi. If this can be accomplished there should be no objection to permitting the use of a greater allowable stress in these high strength steels. The saving in steel resulting from such control should justify the extra effort on the part of the designer.

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†Member American Concrete Institute, Structural Engineer, Structural Engineering and Engineering Mechanics Sections, National Bureau of Standards, Washington, D. C.

‡Member American Concrete Institute, Structural Engineer, Structural Engineering Section, National Bureau of Standards, Washington, D. C.

The purpose of this investigation was to study the mechanism by which cracks form in flexural beams, and to develop formulas that will enable the design engineer to predict the spacing and width of cracks. This analysis is confined to cracks that occur within the region of pure bending on reinforced concrete beams, so that the equations must be modified to make them apply to the shear spans of the beams. Only the cracks due to elastic strain of the reinforcement are considered, and repeated load application or loads with long durations which cause wider cracks are beyond the scope of this paper. Furthermore, since the problem is so complex, simplicity with reasonable rather than rigorous accuracy is the objective. Certain factors which have only minor influence are neglected.

NOTATION

A_s = area of reinforcing steel
A_c = concrete area
A_t = area of concrete affected by the extension of the steel
D = diameter of reinforcing bars
d = effective depth of specimens
E_s = modulus of elasticity of the steel
e = average minimum spacing of cracks
f_t' = tensile strength of concrete
f_s = computed steel stress (based on linear theory)
f_{so} = steel stress just prior to crack formation in concrete
h = total depth of beam specimen
kd = depth of compressive concrete in the specimens
L' = span length of specimen subjected to pure flexure
m = factor determining the diameter of the concrete area affected by the extension of the reinforcing steel
N = number of cracks that occur in the concrete
p = ratio of the steel area to the concrete area
T = tensile force in reinforcing steel at a cracked section
T' = tensile force in reinforcing steel midway between successive cracks
u_o = bond strength of concrete
W_{max} = maximum width of crack at surface of concrete
W_{av} = average width of crack at surface of concrete
$W_s = \dfrac{W_{av}}{\dfrac{h - kd}{d - kd}}$ = average width of crack at the reinforcing steel
ϕ = ratio of the assumed effective area to the fully developed area of concrete
ϵ_s = steel strain
$\sum_o$ = perimeter of reinforcing steel

HISTORICAL BACKGROUND

Investigations prior to World War II concerning cracks in symmetrically reinforced concrete members were summarized by Watstein and Parsons.[1] They found that for a given type of steel at a constant stress level the most prominent factor affecting crack width and spacing was the ratio D/p, and that the crack width was almost completely independent of the concrete strength.

Bornemann[2] studied the mechanism of crack formation in flexural members and concluded that the width of cracks should increase with an increase in the maximum tensile force in the concrete prior to rupture, and decrease with an increase in the ratio of the steel area to the area of the ruptured concrete.

Colonnetti[3] made a theoretical analysis of the mechanism of crack formation in flexural members, and concluded that crack width decreased with a reduction of the diameter of the reinforcing bars.

Watstein and Seese[4] studied the effect of "bond efficiency" of different types of reinforcing bars on the width and spacing of cracks in concrete cylinders subjected to axial tension. They found that the width of the cracks decreased with an increase of bond efficiency, and that at a given steel stress the crack width for various bars varied in a nearly linear relationship with the spacing of the cracks. The latter finding was confirmed by Bjuggren.[5]

Saliger[6] assumed that the concrete between two successive cracks fully participates in resisting the bending moment, and derived the following expressions:

$$e = 0.23\,(D/p)\,\frac{f_t'}{u_o}$$

$$W = k_1\, e\, \epsilon_s$$

where k_1 is a constant dependent on the distribution of steel strains near cracks, and W is the width of cracks. Furthermore, he pointed out that when the tensile strength is lower there will be less adhesive resistance, and in this way the influence of concrete quality is practically eliminated.

Clark[7] modified the Watstein and Parsons[1] equation for average crack width in axially reinforced concrete members under direct tension to make it apply to reinforced concrete members subjected to flexure, incorporating the assumption that crack width is proportional to $(D/p)\ (h-d)/d$ instead of the simple term D/p.

Note that D/p can be transformed into $4A_c/\sum_o$, and the factor $(h-d)/d$ has the effect of reducing the concrete area in the beam to that portion of the concrete immediately below the steel. A somewhat similar conclusion was reached independently by Bjuggren[5] on an earlier date. Bjuggren pointed out that the spacing of cracks in a beam was influenced by the "smallest surrounding area" of concrete.

Wästlund and Jonson[8] made a series of tests on T-beams with various dimensions and steel arrangements. Their findings threw a new light on the cracking problem of flexural members. Some of their conclusions were contrary to previously established ideas and paved the way for a new theory. The major findings in their paper are summarized as follows:

1. For a given stress level in the reinforcement, crack width increased almost linearly with bar diameter for beams of various dimensions and with various amounts of reinforcement.

2. For a given cross section of concrete and diameter of reinforcing bars the width of cracks at the same steel stress in each beam decreased only slightly even when the area of the steel was increased threefold.

3. All other variables being equal, wider beams had considerably larger crack widths, while an increase in beam depth had practically no effect on the width of the cracks.

4. Crack width decreased with an increase in the surface roughness of the reinforcing bars.

5. Variation of the concrete cube strength did not exert any noticeable influence on the width of cracks.

TEST SPECIMENS

Test specimens are described in Table 1 along with the amount of reinforcement and the concrete compressive strengths as determined by tests of 6 x 12-in. cylinders. Specimens of each size which have the same amount of reinforcement differ only in the compressive strength of concrete.

The specimens were cast in steel molds and remained in the molds for 5 days in a damp condition. Then they were subjected to moist curing until 2 days prior to testing at 28 to 30 days. Three 6 x 12-in. cylinders were cast with each specimen and cured under the same conditions as the specimen to determine compressive strength of the concrete.

The reinforcing bars used in this test series were of intermediate grade steel which met the requirements of ASTM A 15-54T for billet steel bars and the deformation of these bars complied with the ASTM A 305-53T requirements.

TABLE 1—DESCRIPTION OF SPECIMENS AND REINFORCEMENT

Specimen No.	Total depth, in.	Effective depth, in.	Width, in.	Span length, in.	Reinforcement		Compressive strength of concrete, psi	ϕ
					Pieces	Bar number		
1	6	5.25	7.5	72	2	4	5840	1.0
2	6	5.25	7.5	72	2	4	2360	1.0
3	6	5.12	11	72	2	6	6110	0.944
4	6	5.12	11	72	2	6	2110	0.944
5	15	13.06	6	108	2	7	6630	0.948
6	15	13.06	6	108	2	7	6655	0.948
7	15	13.06	6	108	2	7	2520	0.948
8	15	13.06	6	108	2	7	2130	0.948
9	15	13.00	6	108	1	8	6100	1.0
10	15	13.00	6	108	1	8	5930	1.0
11	15	13.00	6	108	1	8	3060	1.0
12	15	13.00	6	108	1	8	2285	1.0
13	23	20.87	6	132	1	10	6330	0.969
14	23	20.87	6	132	1	10	2400	0.969
15	23	20.94	6	132	2	9	6460	0.786
16	23	20.94	6	132	2	9	2450	0.786

Concrete used to cast these specimens was proportioned for nominal compressive strengths of 2000 and 6000 psi at 28 days. Type I cement, sand, and gravel were proportioned by weight at approximately 1:3.9:6.1 with 11 gal. water per sack to produce the 2000-psi concrete, and 1:1.4:2.3 with 5 gal. water per sack to produce the 6000-psi concrete. Both sand and gravel were siliceous aggregates and the maximum size of coarse aggregate was 1 in.

Instrumentation for measuring the width of tensile cracks in the specimens consisted of two rows of Tuckerman optical strain gages placed equidistant from the longitudinal center line of the specimen and staggered to overlap each other by 0.5 in. This instrumentation was used to include every crack that formed within the region of constant bending moment. Electrical resistance wire strain gages were applied in pairs on the reinforcing steel at midspan to determine the steel strain. The location of the strain gages and the loading method are shown schematically in Fig. 1.

TESTING PROCEDURE

All of the specimens were placed in the testing machine with the tensile side up and loaded near the ends (Fig. 1). This arrangement offered an easily accessible flat surface for installation of the Tuckerman gages. Rockers were used to support the specimens at their quarter-points and a roller and spherical bearing unit were used to apply the load near the ends. All loads

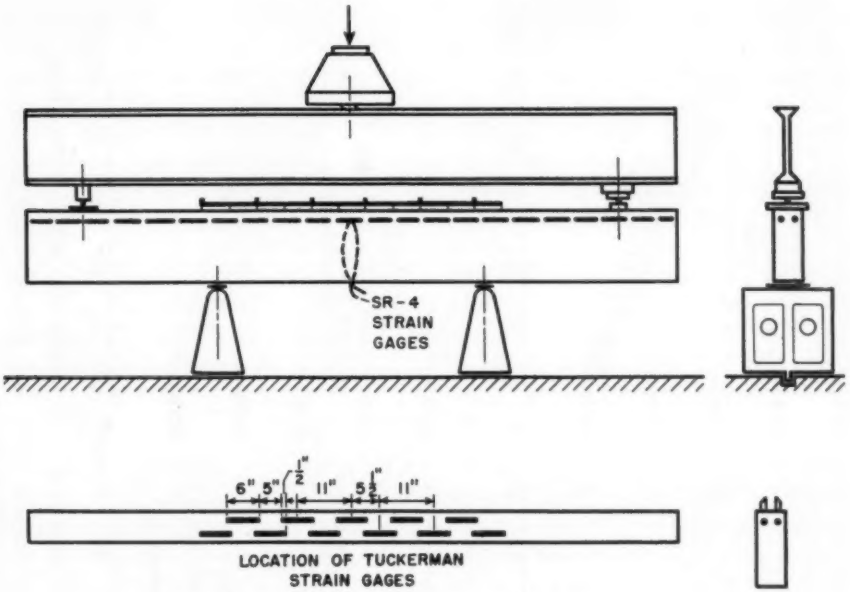

Fig. 1—Instrumentation and loading diagram

and reactions were transmitted to the specimens by steel bearing plates 1 in. thick, 3.5 in. wide, and long enough to cover the full width of specimen.

Zero load readings were made after the specimen was preloaded to 500 lb and the load increments depended on the size of the specimen, the range being from 500 to 5000 lb. Each increment of load was applied at a rate of 1000 lb per min. All gages were read and the location and extent of cracks were recorded immediately after application of each increment.

The extension indicated by the Tuckerman gage was taken to represent the width of crack spanned by the gage, and no attempt was made to correct the crack width for the strain in the concrete. Although the gages indicated tensile strain in the concrete before the cracks formed in the gage length, this strain was small compared to the strain indicated after the cracks appeared. Furthermore, Watstein and Mathey[9] have reported test results which definitely establish that compressive strains can exist on the tensile face of concrete midway between successive cracks. Therefore, due to the random occurrence of cracks and the uncertain magnitude of concrete strain between cracks, it was deemed inadvisable to attempt the correction of the crack width measurements.

TEST RESULTS

This test series of 16 specimens was designed and tested by the late Arthur P. Clark as a continuation of his previous investigation.[7] Specimens identical except for compressive strength of the concrete were used to determine the effect of compressive strength on crack formation. The data presented herein seemed to indicate no appreciable effect for compressive strengths from 2000 to 6000 psi. If there was any effect it was small enough to be veiled by the normal scatter as shown in the six graphs of Fig. 2. Each graph represents the test results of beams of identical design. The black points represent data

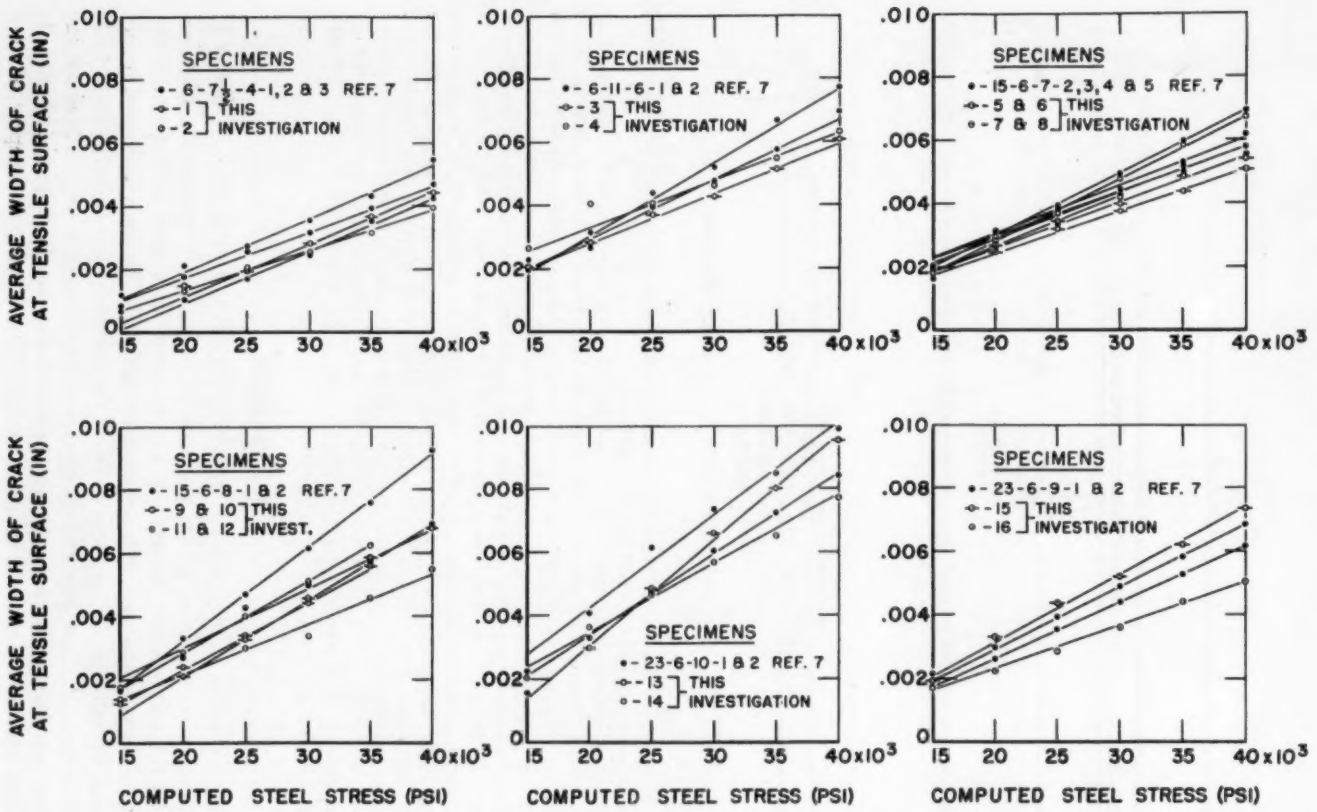
AVERAGE WIDTH OF CRACK AT TENSILE SURFACE (IN)
.010
.008
.006
.004
.002
0
SPECIMENS
• - 6-7½-4-1, 2 & 3 REF. 7
○ - 1] THIS
○ - 2] INVESTIGATION
SPECIMENS
• - 6-11-6-1 & 2 REF. 7
○ - 3] THIS
○ - 4] INVESTIGATION
SPECIMENS
• - 15-6-7-2, 3, 4 & 5 REF. 7
○ - 5 & 6] THIS
○ - 7 & 8] INVESTIGATION
SPECIMENS
• - 15-6-8-1 & 2 REF. 7
○ - 9 & 10] THIS
○ - 11 & 12] INVEST.
SPECIMENS
• - 23-6-10-1 & 2 REF. 7
○ - 13] THIS
○ - 14] INVESTIGATION
SPECIMENS
• - 23-6-9-1 & 2 REF. 7
○ - 15] THIS
○ - 16] INVESTIGATION
15 20 25 30 35 40x10³
COMPUTED STEEL STRESS (PSI)

Fig. 2—Typical sample of observed data; white dots indicate 2000-psi concrete; black dots indicate 4000-psi concrete; and white dot with horizontal line indicates 6000-psi concrete

of beams with a nominal concrete compressive strength of 4000 psi from Reference 7; the open points (white dots) represent data from the beams presented herein with the nominal concrete compressive strengths of 2000 and 6000 psi.

The problem of measurement was further complicated by the likelihood that more than one crack may form in the 6-in. gage length of the Tuckerman gages. Therefore, it was necessary to exercise judgment in deciding which gage readings to use in determining average and maximum widths of cracks.

The results of this test series are summarized in Table 2 showing number of cracks present, average spacing, average width, and maximum width of crack at the various steel stresses as computed from the linear theory. No results are presented for a steel stress over 40,000 psi as this was the minimum yield strength of the steel.

ANALYSIS

Since formation of cracks in the beam is a random phenomenon, the location, spacing and width of cracks are subject to considerable individual variation. Furthermore, as the cracks occur the performance of the specimen becomes an ever changing complex which makes rigorous theoretical analysis extremely difficult. On the other hand, a purely empirical approach may lead to erroneous conclusions when based on test data of limited scope. Therefore, to attain simplicity plus reasonable validity of the equations, a semi-empirical approach was adopted.

This analysis is based on the following assumptions:

1. Concrete is a homogenous and elastic material.
2. The cracked portions of the beams are subjected to pure bending.
3. Reinforcing steel is not stressed past its proportional limit.
4. After a number of cracks have occurred in the tensile zone of the concrete the tensile strains due to flexure are negligible. Any measurable strains in that portion of the concrete are attributed to the shear deformation developed through bond by the extension of the steel.

The first three assumptions are those normally used in crack investigations. Assumption 4 can be easily verified by strain measurements as Clark[7] and Watstein and Mathey[9] have done. As mentioned previously strains on the cracked tensile face of a beam are quite small compared to the crack width and may be of either sign. Therefore, these strains were neglected for simplicity.

It has been pointed out that the deformation of the reinforcing bar causes a shear deformation of the concrete area immediately surrounding the bar. The deformed area is only a portion of the total concrete area of the tensile zone, and the authors consider its relative magnitude an important factor in determining the width and spacing of cracks. The illustration in Fig. 3 (a) is a greatly exaggerated profile of the reinforcing bar and the deformed concrete area.

Considering a semi-infinite elastic body subjected to a uniform load over a portion of its surface, the maximum deflection in the body is not a constant but

TABLE 2—TEST RESULTS

Specimen No.	Number of cracks	Average crack spacing, in.	Average crack width, in.	Maximum crack width, in.	Number of cracks	Average crack spacing, in.	Average crack width, in.	Maximum crack width, in.	Number of cracks	Average crack spacing, in.	Average crack width, in.	Maximum crack width, in.
	f_s = 15,000 psi				f_s = 20,000 psi				f_s = 25,000 psi			
1	2	20.0	0.00083	0.00091	4	10.0	0.00148	0.00191	9	4.5	0.00197	0.00256
2	—	—	—	—	9	3.6	0.00133	0.00247	12	3.3	0.00205	0.00460
3	7	5.1	0.00206	0.00285	9	4.1	0.00286	0.00469	10	3.8	0.00371	0.00659
4	7	5.3	0.00265	0.00375	7	5.0	0.00405	0.00535	10	4.0	0.00404	0.00669
5	10	8.3	0.00170	0.00205	10	5.9	0.00283	—	12	4.9	0.00319	0.00355
6	10	5.7	0.00170	0.00250	13	4.6	0.00250	0.00460	13	4.6	0.00349	0.00499
7	9	5.8	0.00181	0.00323	9	5.8	0.00269	0.00456	10	5.3	0.00367	0.00582
8	16	4.2	0.00184	0.00335	16	4.2	0.00295	0.00522	17	3.8	0.00378	0.00706
9	1	—	0.00141	—	7	7.8	0.00210	0.00334	10	5.6	0.00328	0.00541
10	5	7.9	0.00131	0.00272	10	5.9	0.00238	0.00497	11	5.5	0.00342	0.00736
11	—	—	—	—	—	—	—	—	10	7.0	0.00300	0.00428
12	5	12.3	0.00179	0.00228	10	6.4	0.00284	0.00416	12	5.6	0.00400	0.00527
13	—	—	—	—	10	7.2	0.00298	0.00610	10	7.2	0.00484	0.00878
14	4	31.6	0.00205	0.00286	12	5.7	0.00364	0.00550	13	5.3	0.00468	0.00735
15	15	5.5	0.00191	0.00282	16	5.1	0.00330	0.00469	17	4.8	0.00437	0.00673
16	12	5.3	0.00169	0.00319	17	4.2	0.00222	0.00502	18	4.0	0.00286	0.00659
	f_s = 30,000 psi				f_s = 35,000 psi				f_s = 40,000 psi			
1	10	3.7	0.00283	0.00353	10	3.7	0.00366	0.00449	10	3.7	0.00446	0.00546
2	12	3.3	0.00253	0.00621	12	3.3	0.00317	0.00783	12	3.3	0.00393	0.00945
3	11	3.5	0.00429	0.00805	11	3.5	0.00512	0.00941	12	3.2	0.00605	0.01071
4	10	3.8	0.00464	0.00796	10	3.8	0.00549	0.00929	10	3.8	0.00631	0.01062
5	15	4.2	0.00400	0.00683	15	4.2	0.00490	0.00840	16	3.9	0.00543	0.01022
6	16	3.8	0.00374	0.00630	17	3.6	0.00440	0.00800	17	3.6	0.00510	0.01000
7	11	4.8	0.00420	0.00698	—	—	—	—	—	—	—	—
8	17	3.8	0.00478	0.00895	17	3.8	0.00585	0.01091	17	3.8	0.00674	0.01257
9	10	5.6	0.00440	0.00739	10	5.6	0.00583	0.00991	10	5.6	0.00682	0.01166
10	11	5.5	0.00454	0.00954	11	5.5	0.00558	0.01170	—	—	—	—
11	11	5.8	0.00336	0.00538	12	5.7	0.00456	0.00672	12	5.7	0.00547	0.00811
12	12	5.6	0.00510	0.00637	12	5.6	0.00622	0.00775	—	—	—	—
13	10	7.2	0.00657	0.01108	10	7.2	0.00800	0.01298	10	7.2	0.00956	0.01566
14	13	5.3	0.00567	0.00903	14	4.9	0.00650	0.01068	14	4.9	0.00772	0.01235
15	18	4.5	0.00517	0.00863	18	4.5	0.00622	0.01049	18	4.5	0.00737	0.01274
16	18	4.0	0.00360	0.00821	18	4.0	0.00439	0.00991	18	4.0	0.00506	

is proportional to the diameter D of the loaded area.[10] It seems, therefore, that the deformed area can be replaced by a hypothetical cylindrical area shown in Fig. 3 (b) without causing a major change in the basic equilibrium. The ratio between the diameter of the deformed area of concrete and the diameter of the bar can be determined by direct measurement or by the available crack data. The latter alternative was chosen for this investigation. It is conceivable that the deformed area will be distorted when it intercepts the exterior surface of the beam or the boundary of a similar deformed area around another reinforcing bar in close proximity. This effect can be taken into account approximately by deducting the overlapping areas from the total deformed or fully developed area as indicated in Fig. 3 (c). The ratio of the assumed effective area to the fully developed area is designated as ϕ and written as

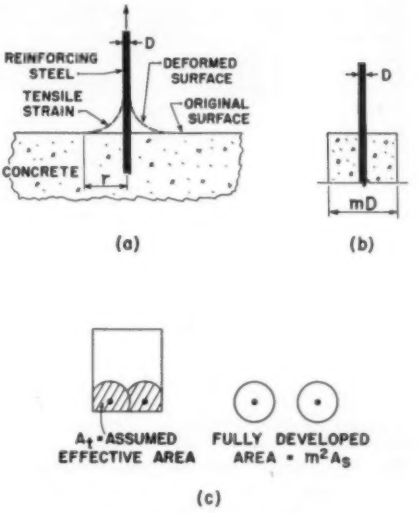

Fig. 3—Effective concrete area

$$\phi = \frac{A_t}{m^2 A_s}$$

The values of ϕ for the test specimens reported herein are given in Table 1.

A typical portion of a beam subjected to pure bending is shown (with the width of the cracks exaggerated) in Fig. 4 (a). It is assumed that the flexural stress in the concrete between successive cracks is negligible. Therefore, the bond stress between the steel and the concrete is one of the determining factors in this investigation. It is evident from the free body diagram in Fig. 4 (b) that the bond stress was assumed to vary linearly from zero at Section 2-2 to the maximum at Section 1-1, and that the tension in the steel T' at Section 2-2 is smaller than T at Section 1-1 by the amount of the shear forces along the bar or

$$T - T' = \frac{u_o e \sum_o}{4} \quad \ldots\ldots (1)$$

The concrete tensile stress at the midsection of the segment of length e due to the direct tension of the bar which is transmitted through bond is

$$A_t f_t' = T - T' \quad \ldots\ldots (2)$$

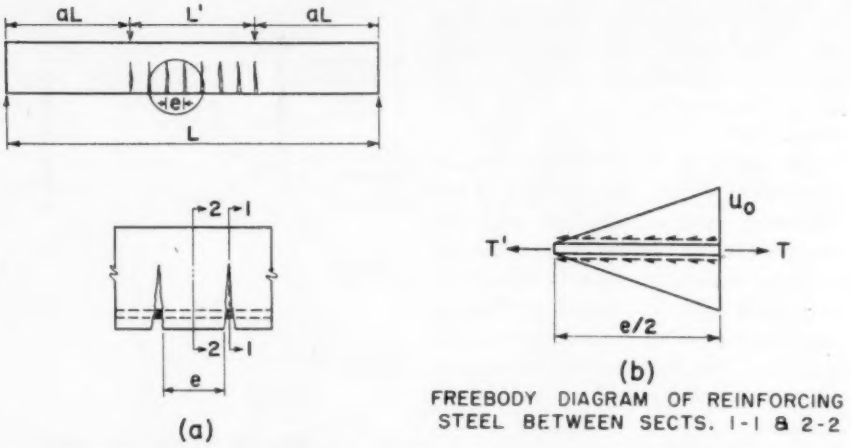

Fig. 4—Flexural member subjected to uniform bending moment over portion of span

Combining Eq. (1) and (2), we have

$$e = \frac{4f_t'}{u_o} \frac{A_t}{\sum_o} \quad \ldots\ldots (3)$$

and by incorporating the definition $\phi = A_t/m^2A_s$ we have

$$e = \frac{f_t'}{u_o} m^2 \phi D \quad \ldots\ldots (4)$$

Again if the concrete strain is neglected we can write

$$\frac{nW_s}{L'} = \frac{f_s - f_{so}}{E_s} \quad \ldots\ldots (5)$$

and when n is large compared to 1

$$L' = ne \quad \ldots\ldots (6)$$

By substituting L' from Eq. (6) and e from Eq. (4) into Eq. (5), we have

$$W_s = \frac{f_t'}{u_o} m^2 \phi D \frac{(f_s - f_{so})}{E_s} \quad \ldots\ldots (7)$$

Eq. (4) and (7), as derived through this simple semi-empirical approach, are in such a form that the constants are easily evaluated empirically. The value of $m = 4$ was estimated through successive approximations, and f_t'/u_o and f_{so} were determined from straight lines fitted to the data shown in Fig. 5 and 6. These data are from the 16 beams reported herein, and from Clark's

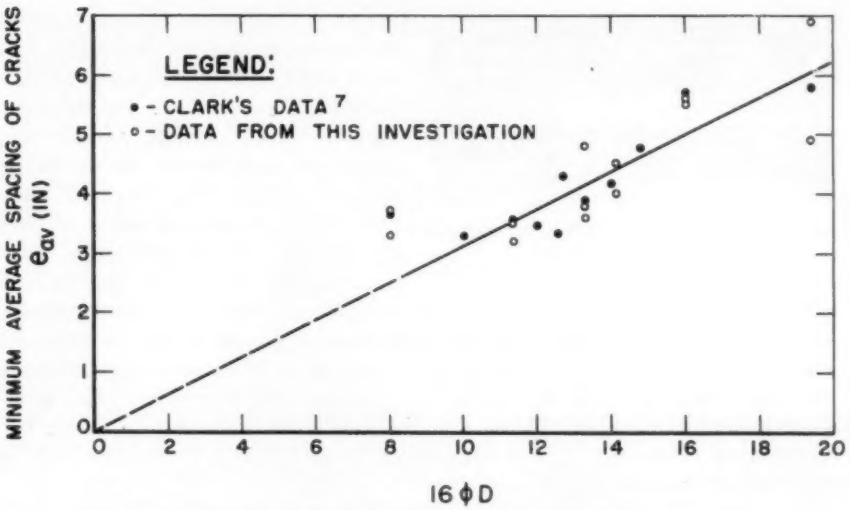

Fig. 5—Experimental results used to determine f_t'/u_o for Eq. (4)

$$e = [f_t'/u_o]\, m^2\, \phi\, D$$

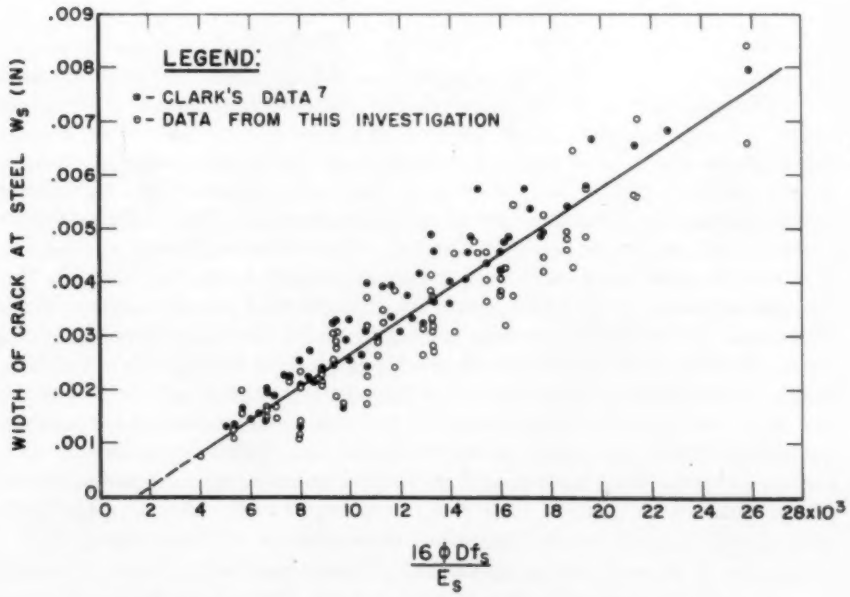

Fig. 6—Experimental results used to determine f_t'/u_o and f_{so} for Eq. (7)

$$W_s = \frac{f_t'}{u_o}\, m^2\, \phi\, D\, \frac{(f_s - f_{so})}{E_s}$$

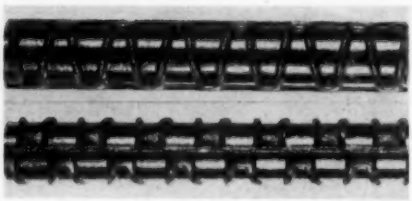

Fig. 7—Deformed bar used in this investigation and by Clark[7]

paper.[7] Clark's previously reported data were selected by discarding data from slabs with percentages of steel less than normally used, and by selecting the larger groups of identical specimens to determine representative average values. The slope of these lines, being the ratio of tensile to bond strength of the concrete, was found to be 5/16. However, this ratio is bound to vary for bars with different deformation patterns or surface roughness. Therefore, to apply these results to various bars one must accept the approach suggested by Brice[11] and apply a correction to the 5/16 slope for the deformed bar used in this study shown in Fig. 7 by determining the relative bond value[4] of other bars with respect to them. The steel stress just prior to crack formation in the concrete, f_{so}, was determined to be $2500/\phi D$ by the intercept of the abscissa shown in Fig. 6.

The incorporation of these empirical values in Eq. (4) and (7) yield

$$e = 5\,\phi\,D \qquad (8)$$

and

$$W_s = 5\,\phi\,D\left[\frac{f_s - \dfrac{2500}{\phi\,D}}{E_s}\right] \qquad (9)$$

Note from Eq. (8) and Fig. 5 that the average minimum spacing of cracks is directly proportional to the product ϕD. This indicates that the arrangement and the diameter of the steel are of equal importance. This is illustrated by Clark's data[7] on beams listed in Table 3. These beams differed in thickness of concrete cover from their respective companion beam specimens. This change produced appreciable effects on the observed crack widths. These effects can be explained by this analysis. Since the fully developed area (m^2A_s) remains unchanged for each set of beams, then from $\phi = A_t/(m^2A_s)$ a change in concrete cover causes a corresponding change in ϕ. From Eq. (8) and (9) it can be seen that the change of concrete cover produces a corresponding change in the computed values of spacing and width of cracks. A comparison between the observed and computed values of spacing and width of cracks is shown in Table 3. The results indicate that the theory predicts the spacing and width of cracks for various thicknesses of concrete cover.

Because of the complex nature of the problem the theory can be expected to apply only to average values of crack width. Although a comprehensive statistical study of the dispersion of the data on widths of cracks was not made, one measure of the variability of the widths of cracks in a beam was obtained by determining the ratios of the maximum to average widths of

TABLE 3—EFFECT OF CONCRETE COVER

Beam	d, in.	Cover, in.	Average crack spacing, in.			Average crack width, in.		
			Computed	Observed	Percent discrepancy	Computed	Observed	Percent discrepancy
15-6-6-1 15-6-6-2 15-6-6-3	13.38 13.13	1.25 1.50	3.73 3.73	3.61 4.29	−3 15	0.00462 0.00476	.00422 .00605	−9 13
15-6-7-1 15-6-7-2 15-6-7-3 15-6-7-4	14.06 13.06	0.50 1.50	3.50 4.12	3.58 3.98	2 −3	0.00398 0.00545	.00411 0.00535	4 −2
15-6-9-1 15-6-9-2 15-6-9-4 15-6-9-5	13.56 12.94	0.88 1.50	3.80 4.40	2.96 3.65	−22 −17	0.00476 0.00610	0.00376 0.00534	−21 −23

cracks for 70 beams at each of six stress levels (15,000 to 40,000 psi computed steel stress). The lengths over which the cracks were measured for these beams ranged from 36 to 66 in. and the ratio of maximum to average width of crack ranged from 1.08 to 2.64.* The average value for this ratio was found to be 1.63, which agrees with the value of 1.64 reported by Clark.[7] The relationship does not necessarily apply to the longer members used in practice.

SUMMARY

1. A new concept is introduced into the analysis of the problem of crack formation in a portion of a beam under pure bending. After some initial cracks have occurred the tensile force in the concrete is resisted by an "effective area" of concrete immediately surrounding the steel, which is less than the total area of the concrete in the tensile zone of the beam. Further cracking of the concrete is attributed to the direct tension developed through bond and due to the strain in the steel.

2. The minimum average spacing of cracks was found to be proportional to the product of the diameter of the reinforcing bar and the parameter ϕ, which is dependent on the general arrangement and diameter of reinforcement. The analysis indicates that the constant of proportionality is a measure of the ratio f_t'/u_o. Watstein and Seese[4] and Brice[11] have shown that a linear correction can be applied to the ratio f_t'/u_o to extend the solution to cover a variety of different deformed bars. This correction is determined from the relative bond values of the bars.

3. The average width of the cracks at the steel is given by the product of the average minimum spacing of the cracks and a function of the computed steel strain.

4. For the lengths of span subjected to pure flexure in these tests (from 36 to 66 in.) the ratio of the maximum to average width of crack was found to range from 1.08 to 2.64 with an average value of 1.63.

5. Test results reported herein indicate that concrete strength in the range from 2000 to 6000 psi has practically no effect on the formation of cracks.

*Beams containing only one or two cracks at a given stress level were not included in these calculations.

ACKNOWLEDGMENTS

The test data were accumulated by the late Arthur P. Clark while he was a research associate of the American Iron and Steel Institute Fellowship at the National Bureau of Standards. Appreciation is also expressed to C. Burton, student trainee, who performed the major portion of numerical computations in this investigation.

REFERENCES

1. Watstein, D., and Parsons, D. E., "Width and Spacing of Tensile Cracks in Axially Reinforced Concrete Cylinders," *Journal of Research*, National Bureau of Standards, V. 31, RP1545, 1943.

2. Bornemann, E., "Increasing the Tensile Strength and Avoidance of Formation of Cracks in Concrete," *Proceedings*, Second Congress, International Association of Bridge and Structural Engineering, 1936.

3. Colonnetti, G., "The Tensile Strength of Stressed Parts in Reinforced Concrete," *Proceedings*, Second Congress, International Association of Bridge and Structural Engineering, 1936.

4. Watstein, D., and Seese, N. A., Jr., "Effect of Type of Bar on Width of Cracks in Reinforced Concrete Subjected to Tension," ACI JOURNAL, Feb. 1945, *Proc.* V. 41, pp. 293-304.

5. Bjuggren, Ulf, discussion of "Bond and Anchorage" by T. D. Mylrea, ACI JOURNAL, Part 2, Dec. 1948, *Proc.* V. 44, pp. 552-1–552-4.

6. Saliger, R., "High-Grade Steel in Reinforced Concrete." *Proceedings*, Second Congress, International Association of Bridge and Structural Engineering, 1936.

7. Clark, Arthur P., "Cracking in Reinforced Concrete Flexural Members," ACI JOURNAL, Apr. 1956, *Proc.* V. 52, pp. 851-862.

8. Wästlund, G., and Jonson, P. O., "Investigation on Formation of Cracks in Reinforced Concrete Structures," *Preliminary Publication*, Third International Congress of Bridge and Structural Engineering, 1948.

9. Watstein, D., and Mathey, R. G., "Evaluation of Width of Cracks in Concrete at the Surface of Reinforcing Steel by Means of Tensile Bond Specimens," RILEM Symposium on Bond and Crack Formation in Reinforced Concrete, Stockholm, June 1957.

10. Timoshenko, S., and Goodier, N., *Theory of Elasticity*, McGraw Hill Book Co., Inc., New York, 1951, p. 368.

11. Brice, L. P., "Théorie de la Fissuration des Piéces en Béton Armé," *Annales*, l'Institut Technique du Batiment et des Travaux Publics, (Paris) June 1952.

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Creep of Plain and Reinforced Concrete*

By P. G. FLUCK† and G. W. WASHA‡

SYNOPSIS

Authors have reviewed English literature on laboratory and field tests of creep of concrete. Rather than abstract each article, they present a general discussion of information regarding the creep behavior and the factors influencing such behavior of plain and reinforced concrete.

A list of 121 references is provided; 25 of these are specifically cited in the paper, but all have been considered in its preparation. This summary of published test results is intended for the general reader and may serve as a starting point for those who wish to continue study of this subject.

PLAIN CONCRETE

Nature of creep§

Under the action of sustained forces, concrete exhibits nonelastic deformation which increases at a decreasing rate during the loading period. This deformation is called creep and is believed due to closure of internal voids, viscous flow of the cement-water paste, crystalline flow in aggregates, and the flow of water out of the cement gel due to external load and drying. The magnitude and rate of creep for most concrete structures are intimately related to the drying rate, but it should be noted that creep is also important in massive structures such as dams where little or no drying of the concrete takes place. In dams it is believed that most of the creep is due to the flow of the absorbed water from the gel (seepage) caused by the application of external pressure.

Effect of constituents

Both composition and fineness of portland cement influence creep characteristics. Concrete made with low-heat cement creeps more than concrete made with normal cement at all ages. In one series of tests,[79] concrete made with normal and with high-early-strength cements approached maximum creep in about 2 years, while concrete made with a low-heat cement reached the same condition in 5 years. Davis and Troxell[115] have suggested that low drying shrinkage may be obtained with ASTM Type I, II, or III cements if the tricalcium silicate content exceeds 45 percent and if maximum amounts

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†Member American Concrete Institute, Professor, Department of Mechanics, University of Wisconsin, Madison, Wis.

‡Member American Concrete Institute, Chairman, Department of Mechanics, University of Wisconsin, Madison, Wis.

§While the term *creep* is used in this paper, the term *plastic flow* has been used by many authors in the past.

of alkali, loss on ignition, and tricalcium aluminate be not more than 0.5, 1.0 and 12.0 percent, respectively. They further suggest that a similar specification would be expected to provide a concrete with low creep values because of the close relationship between drying shrinkage and creep.

Data on the effect of fineness of cement on the creep properties of concrete are scarce, but Davis and Troxell[115] state that fineness is probably not as important as composition. From tests conducted by Davis, Davis, and Brown[65] it appears that for low-heat cement, creep is greater the coarser the cement, but that the reverse is true for normal portland cement.

No great amount of work has been done to determine the effects of admixtures on the creep properties of portland cement concrete. Evidence now available indicates that the use of approved air-entraining agents has no appreciable effect on creep. Concretes made with pozzolans generally exhibit greater creep than concretes made without pozzolans. With other things equal, it appears that creep increases as the percentage of cement replacement increases. Where creep is an important factor, proprietary compounds should not be used unless their effects on shrinkage and creep have been previously determined because of their uncertain reactions when used with different cements and in different mixtures.

The size, grading, and mineral character of the aggregate all have an appreciable effect on creep of portland cement concrete.[35] Under comparable conditions, it appears that shrinkage and creep decrease as the maximum size of coarse aggregate increases, and also that both shrinkage and creep decrease when well graded aggregates with low void contents are used. The mineral character of the aggregate has an important influence on the creep properties of concrete. Hard, dense aggregates with low absorption and a high modulus of elasticity are desirable when concrete with low shrinkage and creep are wanted. Under comparable conditions, it appears that increasing amounts of creep may be expected depending upon the aggregates used, in the following order: limestone, quartz, granite, basalt, and sandstone.

Effect of proportions

In considering the effect of mix proportions on creep, the interrelationships between water content, slump, water-cement ratio, and proportions of constituents must be kept in mind. Tests by various investigators have shown that creep of concrete decreases as the water-cement ratio and the volume of cement paste decrease. In addition, it has been shown that when a constant water-cement ratio is maintained, creep increases as the slump and cement content increase, or essentially as the amount of cement paste is increased. Washa[79] has shown that a hand-rodded concrete with a slump of 2 in. had creep values from 1½ to 2 times as great as those for zero-slump vibrated concrete having the same water-cement ratio.

Effect of curing

Temperature and humidity during the curing period prior to loading have an important effect on creep. The tendency of concrete to creep decreases

as cement hydration increases. Consequently, considering hydration alone, water-cured concrete should creep less than air-cured concrete. It must also be recognized that the humidity and temperature conditions during curing may cause shrinkage or swelling which have a strong influence on creep. Under compressive load preswelled specimens (resulting from moist curing) creep more than preshrunk specimens (resulting from curing in dry air). The effects of hydration and preswelling are thus opposing factors.

Size effects are important in curing since small specimens respond more rapidly than large specimens to moisture changes. Hence, under similar curing conditions, the degree of hydration and the moisture content of large and small specimens may be different at the time of loading.

Effect of storage

Rate of drying during the loading period has a major effect on creep. Rate of drying is determined by the humidity and temperature of the air, size of the member, characteristics of the concrete, and moisture content of the concrete. Tests by many investigators have shown that the rate and ultimate magnitude of creep increase as humidity of the storage air decreases. Compressive creep in air at 50 percent relative humidity is usually more than twice as great as that obtained under moist storage conditions. Temperature during the loading period is also important because of its significant effect on the physical properties of the gel and the water. The rate of moisture loss for a given relative humidity increases with increasing air temperature.

The relation between relative humidity and creep for a given concrete and given compressive load is not linear. Concrete under load in an atmosphere of 100 percent humidity will reach a creep strain approximately equal to the instantaneous strain obtained when the load is first applied. When the humidity is reduced to 70 percent, the ultimate creep is about twice as great, and at a humidity of 50 percent the ultimate creep is about three times as great. This behavior may be explained by noting that the stresses due to shrinkage are superimposed upon those due to the compressive sustained load, and that rapid and nonuniform drying (and shrinkage) cause a redistribution of stress and produce higher compressive stresses near the center of the member.

Protection of concrete members against rapid drying is very beneficial in reducing the rate and ultimate amount of creep. Washa[100] showed that one group of unprotected slabs exhibited about 86 percent of 5-year creep deflection (3.48 in.) during the first half year of sustained loading, while companion specimens sealed with bakelite lacquer and paraffin exhibited about 52 percent of their 5-year creep deflection (2.66 in.) during the same period.

Effect of size

Size of a concrete member is of great importance since large members have greater total frictional resistance to flow along the capillary channels due to the greater distance the water must go to reach the surface, and because less surface area per unit volume of concrete results in slower drying. If the capillaries are partially evacuated, seepage can take place without the ne-

cessity of forcing moisture to the surface under the action of the sustained load. However, if this water is not finally driven to the surface, the amount of creep will be smaller than that for a comparable concrete where complete drying takes place. Consequently, as the size of a concrete member increases, the rate and the ultimate magnitude of creep decrease. It has been estimated that creep of mass concrete may be of the order of one-fourth that obtained with small specimens stored in moist air.

Small sealed laboratory specimens may be used to provide information on the creep properties of mass concrete. The Bureau of Reclamation[112] used sealed cylinders 6 in. in diameter to measure creep of Shasta Dam concrete. Concrete stresses deduced from the creep data were used to compute the total loads carried by the structure. The computed loads checked well with the total known loads.

Effect of age at loading

With a given concrete and a given sustained load, the rate and the magnitude of creep increase as the age at which the sustained load is applied decreases. It is apparent that the ability of concrete to carry sustained loads without excessive creep is a function of cement hydration, and that under given conditions, as cement hydration progresses, the magnitude of creep that will be obtained is decreased.

Effect of time of loading

Investigators have shown that creep increases rapidly during the early portion of the sustained loading period, and that it continues to increase but at a decreasing rate for a long time. In some instances, increases in creep have been reported to 25 years. Approximately one-fourth to one-third of the ultimate creep takes place in the first month of sustained loading, and about one-half to three-fourths of the ultimate creep occurs during the first half year of sustained loading in concrete sections of moderate size. Richart and Jensen[70] have measured creep in concrete in compression in relatively short periods, varying from 1 to 30 min, under various intensities of stress. Evans[86] has shown that under instantaneous loading, much more creep occurs in the first 0.01 sec than in the period from 0.01 sec to 1.0 min.

Hanson[111] and McHenry[88] have correlated the effects of age at loading and the time after loading on creep by means of a three-dimensional creep surface. In this representation, the age at loading and the time after loading are plotted on two axes at right angles and in one plane, while the creep is plotted perpendicular to the plane of the other two axes.

Effect of stress magnitude

Creep has been observed at stresses as low as 1 percent of the ultimate strength. It is approximately proportional to sustained stress within the range of usual working stresses. The tendency to depart from exact proportionality increases as the values of the sustained stress increase. Sustained stresses above the normal working stresses produce creep that increases at

a progressively faster rate as the magnitude of the sustained stress is increased. Performance of the concrete near compressive failure is basically different than creep. At such high loads, L'Hermite[113] showed that the sonic velocity decreases markedly, that Poisson's ratio increases markedly, and that cracking noises are created which are much greater than the background noise in the concrete.

Alternations of high and low sustained stresses over variable periods produce corresponding changes in creep rate. The instantaneous strain due to the first load application is greater than any succeeding value and is also greater than any recovery strain value. Generally the change in the value of the instantaneous deformation due to reloading and the value of the instantaneous deformation recovery due to unloading are not appreciably affected by the number of alternations. Davis, Davis, and Brown[65] have reported the deformations resulting from repeated alternate high and low loads sustained for equal time intervals. For similar stress conditions the residual deformations were approximately the same at the end of a given period regardless of the number of cycles.

Plastic recovery after release of the applied stress is comparatively small and is generally obtained in a short time. McHenry,[88] Hanson,[111] and Raphael[112] have indicated that creep recovery may be equal to the creep strain if the changes in the concrete properties during the loading and recovery periods are taken into account. The concept of complete recovery is important if the principle of superposition of time effects in concrete is used to calculate creep effects.

Type of creep tests

Since concrete is usually used in compression, most information on creep has been obtained for concrete subjected to sustained compressive loading. In addition, creep tests have been performed under sustained tension, bending, bond, torsion, biaxial, and triaxial stress conditions. The same general behavior pattern is, however, evidenced in nearly all cases.

Tests by Davis, Davis, and Brown[65] on concrete subjected to sustained axial tension and axial compression at early ages showed that the rate of tensile creep was much greater than the rate of compressive creep when both were reduced to a stress of 1 psi, but that after several weeks of sustained stress the rate of compressive creep exceeded that of the tensile creep. Ultimate values of the creep in tension and compression were about equal. In the same paper, a comparison of the creep in tension and compression of nonreinforced beams subjected to constant sustained bending moment showed that the early creep of the fibers on the tension side was greater than the corresponding creep on the compression side. In these tests it was also noted that the surface of zero strain remained in substantially the same position during the test.

Duke and Davis,[91] working with concrete under sustained combined stress, showed that lateral pressure ($\sigma_2 = \sigma_3 = 250$ psi) acting on cylinders under a

given axial load (σ_1 = 500 psi) caused a marked reduction in longitudinal creep and caused lateral compressive creep which exceeded the lateral extensions due to the Poisson's ratio effect under axial load only. In their tests, equal triaxial pressures caused the lateral creep to increase for about 1 month, after which it decreased. Specimens subjected to sustained torsion had shear creep values that were about twice the compressive creep values that would have been obtained under a compressive stress of the same magnitude as the applied shear stress.

As a result of some tests of bond creep, Shank[73] found that it was possible for plain round steel bars to move with respect to the concrete without breaking the bond. These tests indicated further that bond creep is independent of the distance between the bars, that it may be as high as 0.04 in. (in a 20-in. gage length), and that it is not proportional to the compressive strength of the concrete. In these tests the bond creep aided in the elimination of the shrinkage stresses.

Mathematical expressions for creep

Many attempts have been made to develop mathematical relationships which could be used to predict creep of concrete subjected to constant and also to variable sustained loading. However, none have been generally accepted or used, and many designers have taken care of creep by using a modified modulus of elasticity, commonly dividing the elastic modulus by two. This simple method holds reasonably well for a large class of concrete problems, but it is not universally valid and may provide misleading results.

Shank[59] suggested the following formula:

$$E_c = C \sqrt[a]{t}$$

based on the assumption that the creep is proportional to the applied stress. In his equation E_c is the creep strain expressed in μ in. per in. per psi of applied stress, t is the time in days, and C and a are constants which must be evaluated for the specific concrete and loading conditions used.

Lorman[77] has proposed the hyperbolic equation

$$E_c = \frac{mt\,\sigma}{n + t}$$

in which E_c is the creep strain in μ in. per in., σ is the stress in psi, t is the time of loading in days, m is a constant called the creep coefficient and is given in μ in. per in. per psi, and n is the creep-time constant measured in days. Lorman also presents equations for the creep-time relation of concrete subjected to variable sustained load.

McHenry[88] suggests that the following equation satisfactorily relates creep under constant sustained stress and time:

$$Q = \alpha (1 - e^{-rt}) + \beta e^{-pK} (1 - e^{-mt})$$

In this equation, Q is creep strain due to unit load, K is the age at time of loading, t is time after loading, and α, β, r, p, and m are constants which must be determined from laboratory tests. McHenry also presents equations for determining creep under conditions of variable sustained stress.

In the analysis of strain measurements in dams, J. M. Raphael[111] used the following equation:

$$E = F(K) \log_e (t + 1)$$

In this equation K is the age in days at loading, $F(K)$ is the creep constant at age K determined experimentally, t is time after loading in days, and E is the creep deformation in μ in. per in. per psi.

The equations presented are typical, but others are also available. The great variety of equations suggests that the complete mathematical analysis of creep still depends on the gathering and evaluation of additional experimental data* by means of which a clearer understanding of the fundamental laws governing this action may be obtained.

Relation of creep to other concrete properties

It would be highly desirable to be able to predict the long-time creep behavior of concrete from other properties determined from short-time tests. Such information available to the designer would result in a better and more economical design. Some advance along these lines has been made, but information now available is not complete and further study is required. Hanson[111] has shown that creep of mass concrete may be predicted when the change of the elastic modulus with time is available. Chang and Kesler[121] have obtained close agreement between experimental creep data and theoretical creep values predicted from the ratio of sustained stress to ultimate strength, the ratio of dynamic modulus to ultimate strength, and the logarithmic decrement.

Stress relaxation measurements

Investigation of stress relaxation of concrete under long-time loading has received only limited study. In stress relaxation tests, a concrete specimen is loaded at a given age to the desired stress. The load is then released as the measured strain exceeds the instantaneous strain and the specimen is thus maintained at constant strain. Control specimens are used to correct for changes in temperature and autogenous volume changes. Chang and Kesler[121] have obtained good agreement for concrete in compression between experi-

*A paper published since the writing of this survey of creep will be of interest. See Ross, A. D., "Creep of Concrete under Variable Stress," ACI JOURNAL, Mar. 1958, *Proc.* V. 54, pp. 739-758—EDITOR.

mental relaxation data and values predicted from a knowledge of the ratio of initial stress to the ultimate strength and of the logarithmic decrement. Hanson[111] has shown that creep functions could be satisfactorily transformed to stress relaxation functions by a process of graphical integration.

Effect of sustained load on properties of concrete

Long periods of sustained loading influence the properties of concrete. Washa and Fluck[105] have shown that the modulus of elasticity of concrete cylinders that had carried sustained load for 10½ years was about 25 percent greater than the modulus of elasticity of companion unloaded specimens at the end of the same period. The effect on ultimate strength was not as definite, since hand-rodded concrete cylinders that had been loaded during the 10½ years were about 5 percent stronger than companion unloaded cylinders, but with vibrated concrete the unloaded cylinders were about 5 percent stronger. Hanson[111] has shown a definite relation between modulus of elasticity at various ages and creep of mass-cured concrete.

Information on the effect of time of loading on Poisson's ratio is inconclusive at the present time. Objective research to determine the nature of the variation under different types of stress is needed.

COLUMNS

General behavior of columns

Many tests have shown that when reinforced concrete columns are subjected to sustained load there is a tendency for additional stress to be gradually transferred to the steel, with a consequent decrease in the concrete stress. Davis, Davis, and Brown[65] report that in reinforced columns, containing 1.9 percent steel, under sustained load for 5½ years in an atmosphere of 50 percent relative humidity, the steel stress increased from about 8000 to 42,000 psi, while concrete stress decreased from 1000 to 300 psi. In the same period, but for columns stored under water, the steel stress increased from about 7000 to 12,000 psi, and the concrete stress decreased from 900 to 800 psi. The increase in steel stress generally varies inversely with the percentage of steel and with humidity of the air. Faber[23] indicates that the compressive steel in columns may be stressed from 3 to 5½ times as high as the calculated values because of the effects of creep and shrinkage.

Tests by Richart and Staehle[40] have shown that the ultimate strength of columns loaded to failure was the same for columns that had been held under sustained loading for a year as for companion columns which had not been loaded. Richart[48] also showed that when reinforced concrete columns were subjected to sustained loads near the ultimate (95 percent of the short-time ultimate) failure occurred in a few hours. Loads of 80 to 90 percent of the ultimate, causing longitudinal strains several times the yield point strain of vertical steel, were carried for several years.

Viest, Elstner, and Hognestad[120] have shown that eccentrically loaded short reinforced concrete columns may be expected to carry a sustained load of about 90 percent of the ultimate load obtained with fast loading regardless of concrete strength and eccentricity. In their tests, the failure load under sustained loading varied between 82.6 and 94.0 percent of the ultimate load of companion specimens tested to failure under fast loading. They concluded that since column strengths are reduced only about 10 percent under long-time loading—a value well within the range of accuracy of design assumptions—safety in ultimate strength design may be based on the ultimate strength equations for fast static loading.

BEAMS AND SLABS

General behavior of simply supported beams and slabs

In most instances, simply supported beams and slabs are reinforced only in tension and have a nonsymmetrical section. Further lack of symmetry occurs because concrete at and near the top is weaker and contains more water than concrete in the lower portion of the beam because of water rise during placement. Consequently beams and slabs that are not loaded will gradually warp because shrinkage is resisted by the steel at the bottom and because the top concrete shrinks more than the bottom concrete. Beams and slabs subjected to sustained loading consequently will be deflected and strained by warping as well as by loading. In some tests, separation of the two effects has been obtained by determining approximate warp effects from measurements of warp beams supported on edge on the floor and not loaded and subtracting these values from the measurements taken on the loaded beams. These results have usually been reported as "net creep" to distinguish them from results not corrected for warpage, generally called "gross creep."

Simply supported beams and slabs subjected to sustained safe design loads have deflection-time and compression strain-time curves that are similar to the creep-time curves for cylinders under compressive loading. The curves rise steeply during the early portion of the loading period and then continue to rise slowly for a relatively long period. The rate and ultimate value are influenced by the various factors previously discussed for plain concrete and by such factors as position and amount of reinforcing steel and ratio of span length to depth of section. Washa[100] reported a total deflection of 7.42 in. after 5 years of loading for slabs designed in accordance with the 1941 ACI Code. The slabs were 3 in. thick, 12 in. wide, and were end-supported on a 17½-ft. span. This deflection was considered to be made up of three separate parts: 2.49 in. due to initial loading; 1.45 in. due to warp;* and 3.48 in. due to creep. The compressive strain in these slabs after 5 years of loading was 2000 μ in. per in., or about six times the strain at 7 days.

Strain at the tensile steel level showed only a small change during the 5-year loading period. The probable reason for the different behaviors of the

*Caused primarily by the shrinkage of the nonsymmetrical section.

top and bottom portions of the slab during the test is that the effects of shrinkage and creep were in the same direction in the top but were opposite to each other in the bottom of the slab. Consequently the compressive strains in the top surfaces increased continually, while the tensile strains in the steel generally remained nearly constant.

Effect of ratio of span length to beam depth (L/D)

Reinforced concrete beams and slabs may be made in complete conformity with the design provisions of the ACI Building Code and yet may not meet the deflection requirements of Section 203 of that code. Unfortunately few deflection tests of reinforced beams or slabs are made; consequently the lack of stiffness of many such members frequently is not known. Trouble does not always occur, however, because relatively few members are required to carry a sustained load that produces maximum live load stresses.

One factor of considerable importance, the L/D ratio, could be included in the code to prevent excessive deflections, strains, and even occasional failures of reinforced concrete beams and slabs. The magnitude of the effect of this ratio is evident in tests by Washa and Fluck[109] who noted that after 2½ years of sustained loading, the gross creep deflection for beams with an L/D ratio of 70 was from four to six times as great as the creep deflection for beams with an L/D ratio of 20.

Effect of compressive reinforcement

Several investigators have shown that the inclusion of arbitrary amounts of compressive steel at the section of maximum moment in simply supported reinforced concrete beams is effective in reducing creep deflections and compressive creep. Washa and Fluck[109] have shown that the inclusion of compressive steel, not required for design strength, equal in amount to the tensile steel reduced the gross creep deflection about one-half. Inclusion of half as much compressive steel caused a reduction of approximately one-third. The compressive creep was reduced about 60 percent by the inclusion of compressive steel equal to the tensile steel, and about 40 percent by the inclusion of half as much compressive steel. From these significant effects it was concluded that compressive reinforcement should be used where a combination of high L/D ratio and sustained load is necessary.

Effect of sustained overload

The amount of information available on this point is quite meager. A few exploratory tests were made by Washa and Fluck[114] in which beams that previously had been subjected to sustained design load for 3 years were loaded to 85 percent of the short-time ultimate strength. Under this loading, additional creep took place for 9 months, at which time the load was increased to 90 percent. At the end of another month the load was increased to 95 percent, and the beams finally failed under further additions of dead load at the end of 11 months of sustained overload. Prior to subjecting the beams to 95 percent of the ultimate load, they had attained total deflec-

tions from three to five times the immediate deflection caused by the design load. Ultimate load capacities of the overloaded beams which failed primarily in tension were nearly the same as those of companion control beams tested at the start of the overload period.

General behavior of continuous beams and slabs

Tests by Washa and Fluck[119] on reinforced concrete two-span beams resting on three level supports provide information on the behavior of continuous beams under constant sustained load. These tests showed that *at the sections of maximum positive moment* the deflection, compressive creep, and tensile creep behaved in a similar manner to values obtained for simply supported beams. However, comparison of the values for simply supported and continuous beams of similar section and span length shows that after 1 year of sustained load: creep deflection of the continuous beams varied between one-third and one-half of the values for the simply supported beams; compressive creep of the continuous beams varied between 60 and 80 percent of the values for simply supported beams; and the tensile creep for both types of beams was small.

At the section of maximum negative moment (above center support), which was reinforced in tension and compression, the tensile and compressive creep increased with time but at a decreasing rate. The compressive reinforcement in the spans (positive moment region) had little effect on the compressive creep at the section of maximum negative moment, but was effective in reducing the tensile creep at that section.

Proving rings used as center supports showed that the load carried by the center support increased with the time of sustained loading until it was from 1 to 5 percent larger than the initial load. The largest increases were obtained for beams containing no compressive steel at the sections of maximum positive moment. As the center reaction increased, the maximum negative moment, which is numerically larger, increased materially, while the maximum positive moment decreased slightly. At first glance this behavior might seem to contradict the statement generally made that creep causes a gradual redistribution of moments in continuous beams, tending to equalize those of opposite sign. The reason for the difference in behavior lies in the fact that at the section of maximum negative moment the beams under test were heavily reinforced in tension and compression, whereas at the section of maximum positive moment less tensile and compressive (if any) reinforcement was used. In other words, creep was greatly restricted at and near the section of maximum negative moment and less restricted at all other sections. Under these conditions, the maximum negative moment and the stresses at that section increased, while values at the section of maximum positive moment decreased.

Creep of prestressed reinforced concrete beams

In this type of construction it is particularly important that drying shrinkage and creep be minimized to avoid large reductions in the amount of pre-

stress. Consequently special care should be taken in the choice of materials used, the design and placement of the concrete, the conditions and length of the curing period, the age at loading, and the conditions of loading. Under poor job conditions the loss in prestress of the steel due to shrinkage and creep may be as high as 50,000 psi, although 20,000 psi is considered a good average value. At present it is generally considered necessary to prestress the steel at least to 120,000 psi, and it is further felt that prestressing to only 50,000 psi is of little value because of the possible effects of shrinkage and creep. A considerable amount of experimentation on sustained loading of prestressed beams and slabs is needed to answer the many questions that have arisen.

CONCLUDING REMARKS

Consideration herein of the effect of various factors on creep of concrete has been based largely on available experimental evidence rather than on an evaluation of existing mathematical theories. Many of the factors affecting creep and the way they affect it are reasonably well known. Consequently a measure of control of creep is available to engineers and architects. In attempting to control creep, it should be remembered that, unlike shrinkage which is generally undesirable, creep may be desirable or undesirable. It is desirable in that it usually tends to promote a better distribution of stresses in many reinforced concrete structures, and undesirable in that excessive deformations and deflections due to it may necessitate repair of reinforced concrete structures.

Control of creep depends on the proper selection and control of raw materials, mix proportions, manufacturing procedures, curing conditions, age of concrete when loaded, magnitude of sustained load, humidity and temperature of surrounding air, amount and position of reinforcing steel, size of member, and ratio of unsupported length to depth of member.

While some of the answers are known, it must be remembered that a large number of questions still remain unanswered; these answers must be obtained by additional experiment and study. This appears particularly true in reinforced concrete structures because of the variability and complexity of structures and also because of the difficulty of controlling variables in construction.

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REFERENCES

1. Woolson, Ira H., "Some Remarkable Tests Indicating 'Flow' of Concrete Under Pressure," *Engineering News*, V. 54, 1905, p. 459.

2. Hatt, W. K., "Notes on the Effect of Time Element in Loading Reinforced Concrete Beams," *Proceedings*, ASTM, V. 7, 1907, p. 421.

3. Abrams, Duff A., "Tests of Bond Between Concrete and Steel," *Bulletin* No. 71, University of Illinois Engineering Experiment Station, Dec. 1913.

4. Abrams, Duff A., "Test of a 40-Foot Reinforced Concrete Highway Bridge," *Proceedings*, ASTM, V. 13, 1913, p. 884.

5. McMillan, Franklin R., "Shrinkage and Time Effects in Reinforced Concrete," *Bulletin* No. 3, University of Minnesota, Mar. 1915.

6. McMillan, Franklin R., "Shrinkage Stresses and Columns of Edison Building," *Engineering News*, V. 73, 1915, p. 502.

7. Fuller, Almon H., and More, Charles C., "Time Tests of Concrete," *Proceedings*, ACI, V. 12, 1916, pp. 302-316.

8. Goldbeck, A. T., and Smith, E. B., "Reinforced Concrete Slab Tests Show Time Effects," *Engineering Record*, V. 73, 1916, p. 282.

9. Goldbeck, A. T., and Smith, E. B., "Tests of Large Reinforced Slabs," *Proceedings*, ACI, V. 12, 1916, pp. 324-334.

10. Janni, A. C., "Method of Designing Rectangular Reinforced Concrete Flat Slab, Each Side of Which Rests on Either Rigid or Yielding Supports," *Transactions*, ASCE, V. 80, 1916, p. 1689.

11. Quirke, T. T., "Deformation of Concrete Under Loading," *Engineering and Contracting*, V. 45, 1916, p. 571.

12. Smith, Earl B., "The Flow of Concrete Under Sustained Load," *Proceedings*, ACI, V. 12, 1916, pp. 317-323.

13. Smith, Earl B., "Concrete Flows Under Sustained Load," *Engineering Record*, V. 73, 1916, p. 329.

14. Turner, C. A. P., "Measuring Flow in Concrete," *Engineering Record*, V. 73, 1916, p. 652.

15. Lord, A. R., "Extensometer Measurements in a Reinforced Concrete Building Over a Period of One Year," *Proceedings*, ACI, V. 13, 1917, pp. 45-60.

16. Smith, Earl B., "The Flow of Concrete Under Sustained Load," *Proceedings*, ACI, V. 13, 1917, pp. 99-102.

17. Hollister, S. C., "Plasticity and Temperature Deformations in Concrete Structures," *Proceedings*, ACI, V. 15, 1919, pp. 127-138.

18. Williams, G. M., "Some Determinations of the Stress-Deformation Relations for Concrete Under Repeated and Continuous Loadings," *Proceedings*, ASTM, V. 20, Part 2, 1920, p. 233.

19. McMillan, Franklin R., "A Study of Column Test Data," *Proceedings*, ACI, V. 17, 1921, pp. 150-181.

20. Crepps, R. B., "Fatigue of Mortar," *Proceedings*, ASTM, V. 23, Part II, 1923, p. 329 (with discussions by Johnson, A. N., Clemmer, H. F., and Goldbeck, A. T.).

21. Richart, F. E., "A Study of Bending Moments in Columns," *Proceedings*, ACI, V. 20, 1924, pp. 495-522.

22. Hatt, W. K., "Researches in Concrete," *Bulletin* No. 24, Purdue University, Nov. 1925.

23. Faber, Oscar, "Plastic Yield, Shrinkage, and Other Problems of Concrete, and Their Effect on Design," *Proceedings*, Institution of Civil Engineers (London), V. 225, Part 1, p. 27.

24. "Measurement of Elastic Strains in Concrete Structures," (reprinted from the Oct. *Technical News Bulletin*, National Bureau of Standards), *Roads and Streets*, V. 68, Nov. 1928, p. 565; also *Concrete*, V. 33, Dec. 1928, p. 37.

25. Davis, Raymond E., "Flow of Concrete Under Sustained Compressive Stress," *Proceedings*, ACI, 1928, V. 24, pp. 303-335.

26. Johnson, A. N., "Strength Characteristics of Concrete," *Public Roads*, V. 9, 1928, p. 177.

27. Davis, R. E., and Troxell, G. E., "Modulus of Elasticity and Poisson's Ratio for Concrete, and the Influence of Age and other Factors upon these Values," *Proceedings*, ASTM, V. 29, Part II, 1929, p. 678.

28. Maney, G. A., "Sustained Loading of Concrete Beams Reinforced with Compressive Steel," *Engineering and Contracting*, V. 68, 1929, p. 329.

29. Davis, R. E., and Davis, H. E., "Flow of Concrete Under Sustained Compressive Stress," *Proceedings*, ASTM, V. 30, Part II, 1930, p. 707.

30. Glanville, W. H., "Shrinkage Stresses," *Building Research Technical Paper* 11, Department of Scientific and Industrial Research, London, 1930.

31. Glanville, W. H., "The Creep or Flow of Concrete Under Load," *Building Research Technical Paper* 12, Department of Scientific and Industrial Research, London, 1930.

32. Maney, G. A., "Shrinkage of Concrete as a Factor in Compressive Steel Stress," *Engineering and Contracting*, V. 69, 1930, p. 234.

33. Straub, Lorenz G., "Plastic Flow in Concrete Arches," *Transactions*, ASCE, V. 95, 1931, p. 613.

34. Brown, R. L., and Clark, E. C., "The Effect of Time Loading Upon the Bond Stress Between Concrete and Steel," *Proceedings*, ASTM, V. 31, Part II, 1931, p. 690.

35. Davis, R. E., and Davis, H. E., "Flow of Concrete Under the Action of Sustained Loads," ACI JOURNAL, Mar. 1931, *Proc.* V. 27, pp. 837-901.

36. Maney, G. A., and Gamet, M. B., discussion of "Flow of Concrete Under the Action of Sustained Loads," ACI JOURNAL, June 1931, *Proc.* V. 27, pp. 1283-1296.

37. Probst, E., "Influence of Rapidly Alternating Loading on Concrete and Reinforced Concrete," *The Structural Engineer* (London), V. 9, Oct. 1931, p. 326.

38. Shank, J. R., "Plastic Flow of Cement Concrete," *Canadian Engineer*, V. 61, 1931, p. 22.

39. "Tests Provide New Data on Concrete Column Design," *Engineering News-Record*, V. 108, 1932, p. 764.

40. Richart, F. E., and Staehle, G. C., "Fourth Progress Report on Column Tests Made at the University of Illinois," ACI JOURNAL, Jan. 1932, *Proc.* V. 28, pp. 279-315.

41. Lyse, Inge, and Kriedler, C. L., "Fourth Progress Report on Column Tests at Lehigh University," ACI JOURNAL, Jan. 1932, *Proc.* V. 28, pp. 317-346.

42. Whitney, Charles S., "Plain and Reinforced Concrete Arches," ACI JOURNAL, Mar. 1932, *Proc.* V. 28, p. 479-519.

43. Fishburn, Cyrus C., and Nagle, John L., "Tests on a Reinforced Concrete Arch of the Arlington Memorial Bridge," Research Paper 609, *Journal of Research*, National Bureau of Standards, Nov. 1933, V. 11, p. 567.

44. Freyssinet, E., "Flow of Concrete Under Load," (Abstracted by Anderegg, F. O.), *Rock Products*, Mar. 25, 1933, V. 36, p. 44.

45. Glanville, W. H., "Creep of Concrete Under Load," *The Structural Engineer* (London), V. 11, 1933, p. 54.

46. Lyse, Inge, "Fifth Report on Column Tests at Lehigh University," ACI JOURNAL, June 1933, *Proc.* V. 29, pp. 433-442.

47. Thomas, F. G., "A Conception of the Creep of Unreinforced Concrete, and an Estimation of the Limiting Values," *The Structural Engineer* (London), V. 11, 1933, p. 69.

48. Richart, F. E., "Reinforced Concrete Column Investigation," Tentative Final Report of ACI Committee 105, ACI JOURNAL, Feb. 1933, *Proc.* V. 29, pp. 275-284.

49. Probst, E., "Plastic Flow in Plain and Reinforced Concrete Arches," ACI JOURNAL, Nov.-Dec. 1933, *Proc.* V. 30, pp. 137-141.

50. Davis, R. E., Davis, H. E., and Hamilton, J. S., "Plastic Flow of Concrete Under Sustained Stress," *Proceedings*, ASTM, V. 34, Part II, 1934, p. 354.

51. Richart, F. E., Brown, R. L., and Taylor, T. G., "The Effect of Plastic Flow in Rigid Frames of Reinforced Concrete," ACI JOURNAL, Jan.-Feb. 1934, *Proc.* V. 30, pp. 181-195.

52. Wilson, W. M., and Kluge, R. W., "Laboratory Tests of Three-Span Reinforced Concrete Arch Bridges with Decks on Slender Piers," *Bulletin* No. 270, University of Illinois, Engineering Experiment Station, Dec. 1934.

53. Lynam, C. G., *Growth and Movement in Portland Cement Concrete*, Oxford University Press, 1934.

54. Wilson, W. M., "Laboratory Tests of Multiple Span Reinforced Concrete Arch Bridges," *Transactions*, ASCE, V. 100, 1935, p. 424.

55. Gilkey, H. J., and Ernst, G. C., "Report of Project Committee on the Use of High Elastic Limit Steel as Reinforcement for Concrete Sustained Loading Tests on Slender Concrete Beams Reinforced with High Elastic Limit Steel," *Proceedings*, Highway Research Board, V. 15, 1935, p. 81.

56. Shank, J. R., "Plastic Flow of Portland Cement Concrete," *Industrial and Engineering Chemistry*, V. 27, 1935, p. 1011.

57. Shank, J. R., "The Plastic Flow of Concrete," *Bulletin* No. 91, Ohio State University, 1935.

58. Wilson, Wilbur M., and Kluge, Ralph W., "Effect of Time Yield in Concrete Upon Deformation Stresses in a Reinforced Concrete Arch Bridge," *Bulletin* 275, University of Illinois Engineering Experiment Station, 1935.

59. Shank, J. R., "The Mechanics of Plastic Flow of Concrete," ACI JOURNAL, Nov.-Dec. 1935, *Proc.* V. 32, pp. 149-180.

60. Fridenson, B., Gilkey, H. J., and Ernst, G. C., discussion of "Mechanics of Plastic Flow of Concrete," ACI JOURNAL, May-June 1936, *Proc.* V. 32, pp. 704-711.

61. Jones, Paul G., and Richart, F. E., "The Effect of Testing Speed on Strength and Elastic Properties of Concrete," *Proceedings*, ASTM, V. 36, Part II, 1936, p. 380.

62. Ross, A. D., "The Deformation of Concrete Under Load," *The Surveyor*, V. 90, 1936, p. 629.

63. Thomas, F. G., "Cracking in Reinforced Concrete," *The Structural Engineer* (London), V. 14, July 1936, p. 298.

64. "Effect of Plastic Flow and Volume Changes on Design," Report of ACI Committee 313, ACI JOURNAL, Nov.-Dec. 1936, *Proc.* V. 33, pp. 123-128.

65. Davis, R. E., Davis, H. E., and Brown, E. H., "Plastic Flow and Volume Changes of Concrete," *Proceedings*, ASTM, V. 37, Part II, 1937, p. 317.

66. Ross, A. D., "Concrete Creep Data," *The Structural Engineer* (London), V. 15, 1937, p. 314.

67. Shank, J. R., "Tests on Shrinkage of Concrete," *Civil Engineering* (London), V. 32, 1937, p. 264.

68. Blanks, R. F., Meissner, H. S., and Rawhouser, C., "Cracking in Mass Concrete," ACI JOURNAL, Mar.-Apr. 1938, *Proc.* V. 34, pp. 477-496.

69. Davis, R. E., Brown, E. H., and Kelly, J. W., "Some Factors Influencing the Bond Between Concrete and Reinforcing Steel," *Proceedings*, ASTM, V. 38, Part II, 1938, p. 394.

70. Jensen, R. S., and Richart, F. E., "Short-Time Creep Tests of Concrete in Compression," *Proceedings*, ASTM, V. 38, Part II, 1938, p. 410.

71. Powers, T. C., "Measuring Young's Modulus of Elasticity by Means of Sonic Vibrations," *Proceedings*, ASTM, V. 38, Part II, 1938, p. 460.

72. Richart, F. E., and Heitman, R. H., "Tests of Reinforced Concrete Columns Under Sustained Loading," ACI JOURNAL, Sept. 1938, *Proc.* V. 35, pp. 33-40.

73. Shank, J. R., "Bond Creep and Shrinkage Effects in Reinforced Concrete," ACI JOURNAL, Nov. 1938, *Proc.* V. 35, pp. 81-92.

74. Glanville, W. H., and Thomas, F. G., "Further Investigations on the Creep or Flow of Concrete Under Load," *Building Research Technical Paper* 21, Department of Scientific and Industrial Research, London, 1939.

75. Shank, J. R., "Creep of Concrete Reinforcing Bars at Splices," *Civil Engineering* (London), V. 34, 1939, p. 49.

76. Carlson, Roy W., "Attempts to Measure the Cracking Tendency of Concrete," ACI JOURNAL, June 1940, *Proc.* V. 36, pp. 533-540.

77. Lorman, W. R., "The Theory of Concrete Creep," *Proceedings*, ASTM, V. 40, 1940, p. 1082.

78. Maney, G. A., and Lagaard, M. B., "Stress Increases in Compressive Steel Under Constant Load Caused by Shrinkage," ACI JOURNAL, June 1940, *Proc.* V. 36, pp. 541-552.

79. Washa, G. W., "Comparison of the Physical and Mechanical Properties of Hand Rodded and Vibrated Concrete Made with Different Cements," ACI JOURNAL, June 1940, *Proc.* V. 36, pp. 617-648.

80. Maney, G. A., "Plastic Flow of Concrete Declared to be Non-Existent" (abstract), *Concrete*, Apr. 1941, V. 49, p. 33.

81. Grinter, L. E., and Kepford, B., "Long-Time Overload Tests of a T-Beam Floor Panel," ACI JOURNAL, Feb. 1941, *Proc.* V. 37, pp. 433-440.

82. Maney, G. A., "Concrete Under Sustained Working Loads; Evidence that Shrinkage Dominates Time Yield," *Proceedings*, ASTM, V. 41, 1941, p. 1021.

83. Allin, Russell, "Creep In Concrete Under Stress," *Civil Engineering* (London), V. 37, 1942, p. 123.

84. "Strain Meters to Measure Plastic Flow in Shasta Dam," *Engineering News-Record*, V. 128, 1942, p. 425.

85. Pickett, Gerald, "The Effect of Change in Moisture-Content of the Creep of Concrete Under a Sustained Load, "ACI JOURNAL, Feb. 1942, *Proc.* V. 38, pp. 333-356.

86. Evans, R. H., "Some New Facts Concerning Creep in Concrete," *Concrete and Constructional Engineering* (London), V. 37, 1942, p. 429.

87. Davis, R. E., and Kelly, J. W., "Volume Changes and Plastic Flow of Concrete," *Report on Significance of Tests of Concrete and Concrete Aggregates* (ASTM Committee C-9), 1943, p. 54.

88. McHenry, Douglas, "A New Aspect of Creep in Concrete and its Application to Design, *Proceedings*, ASTM, V. 43, 1943, p. 1069.

89. Ross, A. D., "Creep and Shrinkage in Plain, Reinforced, and Prestressed Concrete, A General Method of Calculation," *Journal*, Institution of Civil Engineers (London), V. 21, Nov. 1943, p. 38.

90. Schorer, H., "Prestressed Concrete, Design Principles and Reinforcing Units," ACI JOURNAL, June 1943, *Proc.* V. 39, pp. 493-528.

91. Duke, C. M., and Davis, H. E., "Some Properties of Concrete Under Sustained Combined Stresses," *Proceedings*, ASTM, V. 44, 1944, p. 888.

92. Arnstein, A., and Reiner, M., "Creep of Cement, Cement-Mortar, and Concrete," *Civil Engineering* (London), Sept. 1945, V. 40, p. 198.

93. Shank, J. R., "Creep of Concrete at High Overload," *Ohio State University Engineering Experiment Station News*, Oct. 1945, p. 13.

94. Pickett, Gerald, "Shrinkage Stresses in Concrete," (Parts I and II), ACI JOURNAL, Jan. 1946, Feb. 1946, *Proc.* V. 42, pp. 165-204 and 361-400.

95. Ross, A. D., "The Effects of Creep on Instability and Indeterminacy Investigated by Plastic Models," *The Structural Engineer* (London), V. 24, 1946, p. 413. (Discussion V. 25, 1947, p. 179.)

96. Staley, Howard R., and Peabody, Dean, Jr., "Shrinkage and Plastic Flow of Prestressed Concrete," ACI JOURNAL, Jan. 1946, *Proc.* V. 42, pp. 229-244.

97. Boguslavsky, B. W., "Simplified Formula is Based on Effect of Plastic Flow in Reinforced Concrete," *Civil Engineering*, V. 17, Feb. 1947, p. 87.

98. Seed, H. B., "Creep and Shrinkage in Reinforced Concrete Structures," *Engineering* (London), V. 164, 1947, pp. 309, 375, and 496.

99. Magnel, Gustave, "Creep of Steel and Concrete in Relation to Prestressed Concrete," ACI JOURNAL, Feb. 1948, *Proc.* V. 44, pp. 485-500.

100. Washa, George W., "Plastic Flow of Thin Reinforced Concrete Slabs," ACI JOURNAL, Nov. 1947, *Proc.* V. 44, pp. 237-260.

101. Blanks, R. F., and McHenry, D., "Plastic Flow of Concrete Relieves High-Load Stress Concentrations," *Civil Engineering*, V. 19, May 1949, p. 40.

102. Shank, J. R., "Plastic Flow of Concrete at High Overload," ACI JOURNAL, Feb. 1949, *Proc.* V. 45, pp. 493-500.

103. Davis, R. E., "A Review of Pozzolanic Materials and Their Use in Concretes," *ASTM Special Technical Publication* No. 99, 1949.

104. Hadley, Homer M., "When Concrete Becomes Discrete," *Civil Engineering*, V. 20, Apr. 1950, p. 29.

105. Washa, George W., and Fluck, Paul G., "Effect of Sustained Loading on Compressive Strength and Modulus of Elasticity of Concrete," ACI JOURNAL, May 1950, *Proc.* V. 46, pp. 693-700.

106. Price, Walter H., "Factors Influencing Concrete Strength," ACI JOURNAL, Feb. 1951, *Proc.* V. 47, pp. 417-432.

107. Ross, A. D., "Shrinkless and Creepless Concrete," *Civil Engineering* (London), V. 46, 1951, p. 853.

108. Davis, R. E., and Troxell, G. E., "Properties of Concrete and Their Influence on Prestress Design," presented at Western Conference on Prestressed Concrete, Los Angeles, California, Nov. 14, 1952.

109. Washa, G. W., and Fluck, P. G., "The Effect of Compressive Reinforcement on the Plastic Flow of Reinforced Concrete Beams," ACI JOURNAL, Oct. 1952, *Proc.* V. 49, pp. 89-108.

110. Ban, Shizuo, and Okada, Kujoshi, "Plastic Flow of Reinforced Concrete," *Memoirs of the Faculty of Engineering*, Kyoto University, Kyoto, Japan, V. 15, No. 2, Apr. 1953.

111. Hanson, J. A., "A 10-Year Study of Creep Properties of Concrete," *Concrete Laboratory Report* No. SP-38, U. S. Bureau of Reclamation, 1953.

112. Raphael, J. M., "The Development of Stresses in Shasta Dam," *Transactions*, ASCE, V. 118, 1953, p. 289.

113. L'Hermite, R., "Present Day Ideas on Concrete Technology," *Bulletin* No. 18, Union of Testing and Research Laboratories for Materials and Structures (RILEM, Paris), June 1954.

114. Washa, G. W., and Fluck, P. G., "The Effect of Sustained Overload on the Strength and Plastic Flow of Reinforced Concrete Beams," ACI JOURNAL, 1954, Vol. 50, p. 65. Sept. 1953, *Proc.* V. 50, pp. 65-72.

115. Davis, R. E., and Troxell, G. E., "Properties of Concrete and Their Influence on Prestress Design," ACI JOURNAL, Jan. 1954, *Proc.* V. 50, pp. 381-392.

116. "Review of Literature on Creep in Concrete," *Miscellaneous Paper* No. 6-132, Waterways Experiment Station, Corps of Engineers, 1955.

117. Neville, A. M., "Theories of Creep in Concrete," ACI JOURNAL, Sept. 1955, *Proc.* V. 52, pp. 47-60.

118. Washa, G. W., "Volume Changes and Creep," *ASTM Special Technical Publication* No. 169, "Significance of Tests and Properties of Concrete and Concrete Aggregates," 1956.

119. Washa, G. W., and Fluck, P. G., "Plastic Flow (Creep) of Reinforced Concrete Continuous Beams," ACI JOURNAL, Jan. 1956, *Proc.* V. 52, pp. 549-562.

120. Viest, I. M., Elstner, R. C., and Hognestad, E., "Sustained Load Strength of Eccentrically Loaded Short Reinforced Concrete Columns," ACI JOURNAL, Mar. 1956, *Proc.* V. 52, pp. 727-756.

121. Chang, T. S., and Kesler, C. E., "Correlation of Sonic Properties of Concrete with Creep and Relaxation," *Proceedings*, ASTM, V. 56, 1956, pp. 1257-1272.

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Title No. 54-50

Specific Surface of Aggregates Related to Compressive and Flexural Strength of Concrete*

By B. G. SINGH†

SYNOPSIS

Concrete strength depends on both W/C and specific surface of aggregate. For constant mix proportions, increased specific surface causes decrease in amount of cement relative to surface of aggregate, leaving more voids around surface of aggregate particles. With other mix proportions held constant, as specific surface increased both compressive strength and modulus of rupture tested lower at any given W/C. When compressive strength is plotted against ϕ_c, a factor based on W/C, specific surface of aggregate, and aggregate-cement ratio, a single curve is obtained. A similar factor ϕ_r is developed for flexural strength. A further adjustment is required, in part related to the greater absorption of water in the leaner mixes.

INTRODUCTION

Duff Abrams[1] and many after him have upheld the idea that the strength of fully compacted concrete depended on its water-cement ratio, and that aggregate grading was important only so far as it influenced the water-cement ratio required to achieve full compaction. In other words, provided full compaction is achieved (by whatever method), a given water-cement ratio will give concrete of uniform strength.

Though the idea that the strength of concrete depended solely on its water-cement ratio was an important step in the advancement of concrete technology, good aggregate grading was defined as that having a relatively small variation in specific surface between the extremes for any given type of classification. Under such conditions the effect of grading on strength was not considered significant, and the decrease in strength with finer sands (i.e., higher specific surfaces) was usually explained as due to the increased water-cement ratio required to maintain a given consistency under such conditions.

Apparently, when strengths lower than those expected from a water-cement ratio-strength relationship occur, the results are usually attributed to inadequate compaction. A notable exception to this explanation was advanced by Weymouth[2] with his theory of particle interference in which he argued that loss of strength was due to particle interference resulting from unsuitable aggregate grading. His theory, however suffered from many important limitations and as a result was not much used.

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†Member American Concrete Institute, Research Engineer, Works and Hydraulics Department, Trinidad, B.W.I.

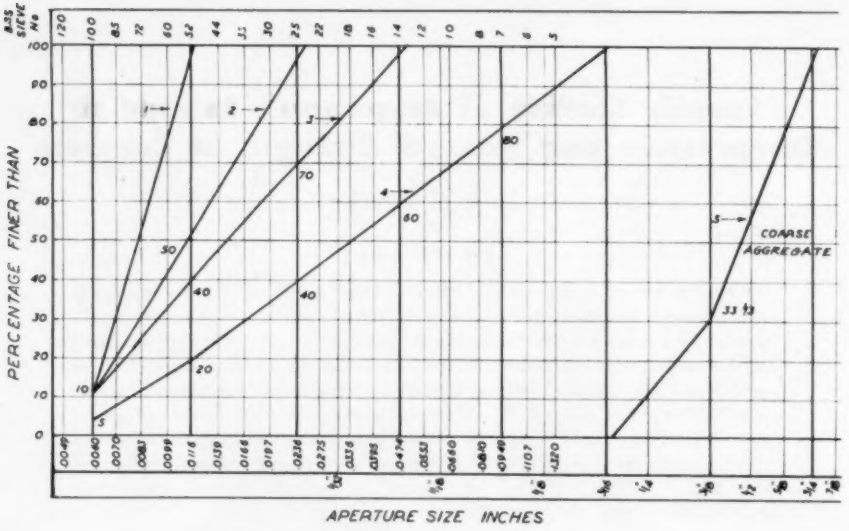

Fig. 1—Grading of fine and coarse aggregate

The strength of concrete determined by any standard method must necessarily be influenced by several factors such as aggregate shape, mineral type, chemical cleanliness, and hardness, and the aggregate surface to be bonded in relation to the available cement matrix, apart from the chemical and physical properties of the cement used, curing, etc. While water-cement ratio and full compaction will always remain the two most significant factors influencing strength, the effect of grading and other aggregate properties on strength cannot be ignored.

The present investigation attempts to ascertain the effect of aggregate grading on the compressive and flexural strength of concrete for practical mixes and consistencies.

The specific surface of the aggregate as measured by the water permeability method[3] is used as an index of grading.

METHOD OF INVESTIGATION

The aggregate throughout the investigation was a normal pit gravel largely of flint and quartzite particles of very low absorption. The aggregate is classified as irregular to rounded as described in British Standard Specification No. 12. The aggregate was perfectly dry before use.

Four fine aggregate gradings and one coarse aggregate grading (see Fig. 1) were made up artificially. Each fine aggregate grading was combined with coarse aggregate in the ratio of 3:7, 4:6, and 5:5. Thus in all 12 combined aggregate gradings of ¾ in. maximum size were used for most of the investigation.

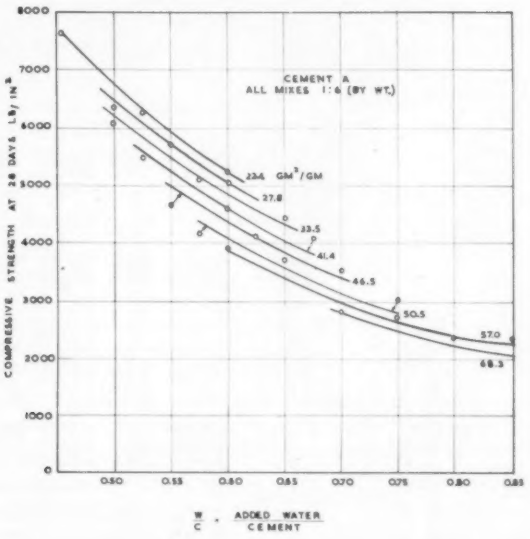

Fig. 2—Effect of grading on the compressive strength. As the specific surface is increased the compressive strength at given W/C ratio decreases

All proportioning was by weight. Though principally 1:4½, 1:6, and 1:7½ mixes were used, 1:3 and 1:9 were also included. A single batch of blended cement stored in airtight containers was used throughout the main experiment. Mixing time was 2½ min.

The mixes were fully compacted by hand ramming, and when necessary with a hammer-type vibrator. The specimens were 4-in. cubes cast in steel molds for compression testing, and 16 x 4 x 4-in. beams for flexural tests. The specimens were finished with a steel float and were stored at 92 percent relative humidity for 24 hr and afterward in water at constant temperature until testing. The air temperature through all other experimental stages was about 64 F.

The rate of loading of the cubes was 2000 psi per min. The average of at least four results for cubes at 7 and 28 days are given. Three 16 x 4 x 4-in. beams were tested on their sides by means of a centrally applied load on a 12-in. span at 7 and 28 days. The central load was applied at the rate of 355 lb per min.

EFFECT OF SPECIFIC SURFACE ON COMPRESSIVE STRENGTH

Fig. 2 shows compressive strength for 1:6 concretes with specific surface of aggregate varying from 22.4 sq cm per g to 68.3 sq cm per g. Absorption value of the aggregate remained practically constant. As specific surface increased compressive strength at a given water-cement ratio decreased. All other mixes tested showed the same characteristics.[4] Experiments summarized in Table 1 support the idea that if the specific surface, water-cement ratio, and mix proportions are constant, the strength is also sensibly constant, irrespective of the grading.

TABLE 1—STRENGTH OF CONCRETES MADE WITH AGGREGATES HAVING A CONSTANT SPECIFIC SURFACE OF 32.0 SQ CM PER G

Cement	Cement-aggregate ratio	Aggregate							Fineness modulus	Water-cement ratio	Density of fresh concrete, lb per cu ft	7-day compressive strength, psi	28-day compressive strength, psi	28-day modulus of rupture, psi
		Grading No.	Grading, percent retained between given sieves*											
			No. 100 and No. 52	No. 52 and No. 25	No. 25 and No. 14	No. 14 and No. 7	No. 7 and $\frac{1}{4}$ in.	$\frac{1}{4}$ in. and $\frac{3}{8}$ in.	$\frac{3}{8}$ in. and $\frac{3}{4}$ in.						
A	1:6	1	25.4	—	—	—	—	74.6		5.41	0.575	146.6	3380	4650	603
A	1:6	2	15.4	15.4	—	—	—	34.6	34.6	4.96	0.575	145.6	3580	4760	650
A	1:6	3	12.9	12.9	12.9	—	—	30.6	30.7	4.76	0.575	145.1	3510	4690	651
A	1:6	4	11.7	11.7	11.7	11.7	—	26.6	26.6	4.63	0.575	145.0	3380	4480	651
A	1:6	5	11.2	11.2	11.2	11.2	11.2	22.0	22.0	4.54	0.575	145.0	3440	4840	636
A	1:6	6	23.4	—	—	5.8	—	21.2	49.6	5.15	0.575	146.2	3660	5050	682
A	1:6	7	24.0	—	—	—	—	38.0	38.0	5.18	0.575	146.5	3580	4820	638
E	1:6	8	18.5	—	9.2	9.2	9.2	26.9	27.0	4.78	0.550	147.3	3820	5520	723
E	1:6	9	17.1	—	17.1	—	17.1	24.4	24.4	4.71	0.550	146.0	4200	5320	—
E	1:6	9	17.1	—	17.1	—	17.1	24.4	24.4	4.71	0.550	145.6	4040	5560	—
E	1:6	1	25.4	—	—	—	—	—	74.6	5.41	0.550	147.7	3550	4940	678
E	1:6	2	15.4	15.4	—	—	—	34.6	34.6	4.96	0.550	146.4	4080	5270	702
E	1:6	3	12.9	12.9	12.9	—	—	30.6	30.6	4.76	0.550	146.4	3830	5120	681
E	1:6	4	11.7	11.7	11.7	11.7	—	26.6	26.6	4.63	0.550	146.3	3940	5370	726
E	1:6	5	11.2	11.2	11.2	11.2	11.2	22.0	22.0	4.54	0.550	146.3	3860	5340	676
E	1:6	7	24.0	—	—	—	—	38.0	38.0	5.18	0.550	147.6	3930	5300	713
E	1:4.5	8	18.5	—	9.2	9.2	9.2	26.9	27.0	4.78	0.550	147.9	4880	6390	815
E	1:4.5	9	17.1	—	17.1	—	17.1	24.4	24.4	4.71	0.550	146.7	5150	7030	—
E	1:4.5	1	25.4	—	—	—	—	—	74.6	5.41	0.550	147.4	4760	6410	695
E	1:4.5	2	15.4	15.4	—	—	—	34.6	34.6	4.96	0.550	147.2	4930	6540	800
E	1:4.5	3	12.9	12.9	12.9	—	—	30.6	30.7	4.76	0.550	147.4	4890	6580	808
E	1:4.5	4	11.7	11.7	11.7	11.7	—	26.6	26.6	4.63	0.550	145.8	5070	6500	805
E	1:4.5	5	11.2	11.2	11.2	11.2	11.2	22.0	22.0	4.54	0.550	146.4	5100	6580	815
E	1:4.5	7	24.0	—	—	—	—	38.0	38.0	5.18	0.550	147.8	4830	6300	810

*British standard sieve numbers.

Fig. 3 shows that density of concrete decreases with increases in the aggregate specific surface for constant aggregate-cement and water-cement ratios. To check whether the decrease in strength and density were due to incomplete compaction, a few mixes were vibrated for a longer period. It was found that density remained practically unaffected though strength tended to decrease.

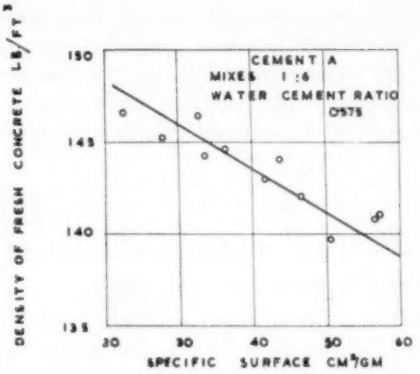

Fig. 3—Density decreases as the specific surface of the aggregate increases for constant water-cement ratio and mix proportions (results plotted were obtained by interpolation)

It would appear from these results that the strength of concrete is affected by both the water-cement ratio and by the specific surface of the aggregate. For constant mix proportions, the increase in the specific surface of the aggregate causes a decrease in the amount of cement relative to the surface of the aggregate, thus causing more voids around the surface of the aggregate particles and a decrease in strength.

The strength of concrete for a constant aggregate-cement ratio, can thus be considered as influenced by:

(a) The water-cement ratio

(b) The area of aggregate per unit weight of cement

For an aggregate of a given specific surface, a rich mix will have a lower area per unit of cement in the ratio N_1: N_2 where N_1 and N_2 are the aggre-

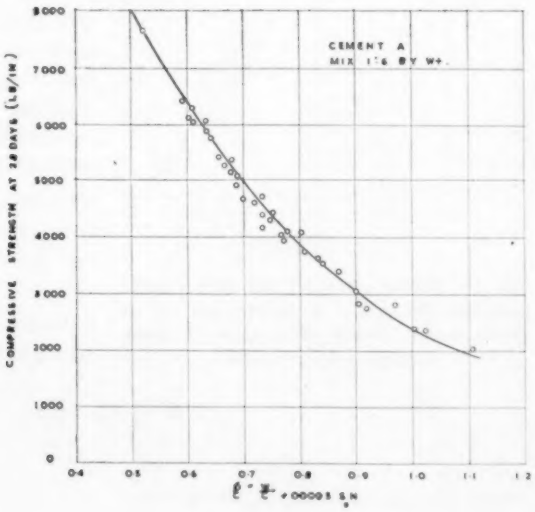

Fig. 4—ϕ_c versus compressive strength for various values of S_o and for $N = 6$

gate-cement ratios for the rich and the lean mix respectively. Therefore, if the area per unit weight of cement is to be taken into account the aggregate-cement ratio N must be also included. Considering the strength of concrete as related to both water-cement ratio and area of aggregate per unit of cement, the following equation can be used.

$$\phi_c = W/C + K_c S_o N \quad \ldots\ldots (1)$$

ϕ_c = number related to compressive strength
W = weight of water in the mix
C = weight of cement in the mix
S_o = specific surface of the aggregate, sq cm per g
N = aggregate-cement ratio (by weight)
K_c = a constant of proportionality

The value of K_c has been found to be of the order of 0.0005 to eliminate the scatter shown in Fig. 2 and to obtain a single ϕ_c versus strength curve shown in Fig. 4.

EFFECT OF MIX PROPORTIONS ON THE STRENGTH OF CONCRETE

Tests show that separate curves are obtained on the water-cement ratio versus strength graph for each mix proportion. Richer mixes at a given water-cement ratio give lower strength for similar aggregate grading and type of aggregate (see Fig. 5). This general tendency is shown by all the tests carried out.

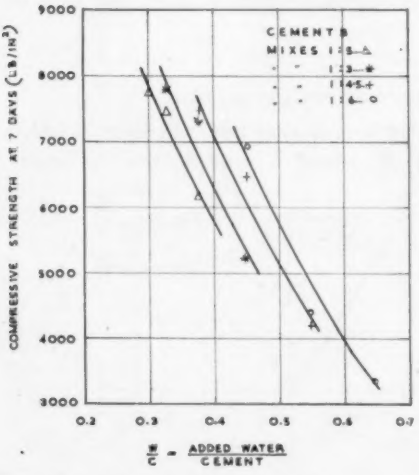

Fig. 5—Water-cement ratio versus compressive strength. A richer mix at a given water-cement ratio gives a lower compressive strength

This is usually attributed to the absorption of water by the aggregate, and since relatively more absorption will occur in the leaner mixes, there will be less water available to the cement, thus giving an *effective* water-cement ratio of a lower value which results in a higher strength. Therefore, in order to obtain a single curve for the various mix proportions, the effective water-cement ratio which excludes absorbed water may be considered in the following form

$$\frac{W^1}{C} = \frac{W}{C} - \frac{KA}{C} \quad \ldots\ldots (2)$$

where K = coefficient of absorption.

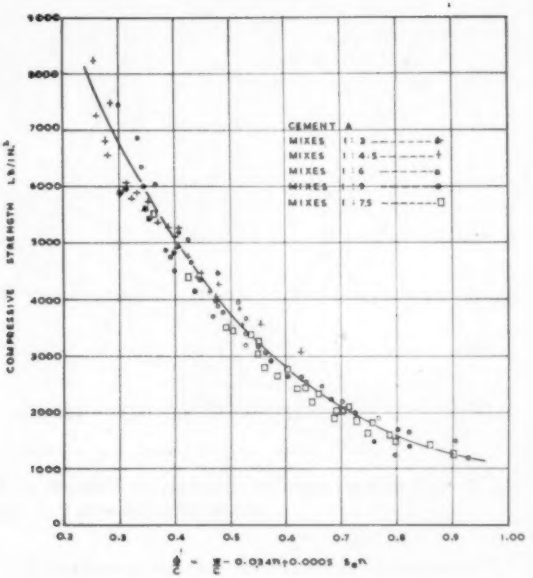

Fig. 6—ϕ_c^1 plotted against strength at 7 days for water-cured concrete of various mix proportions and aggregate of various specific surfaces

Combining Eq. (1) and (2) since both the effective water-cement ratio and the aggregate area per unit weight of cement affect strength, we have:

$$\phi_c^1 = W/C - KN + 0.0005\, S_a N \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

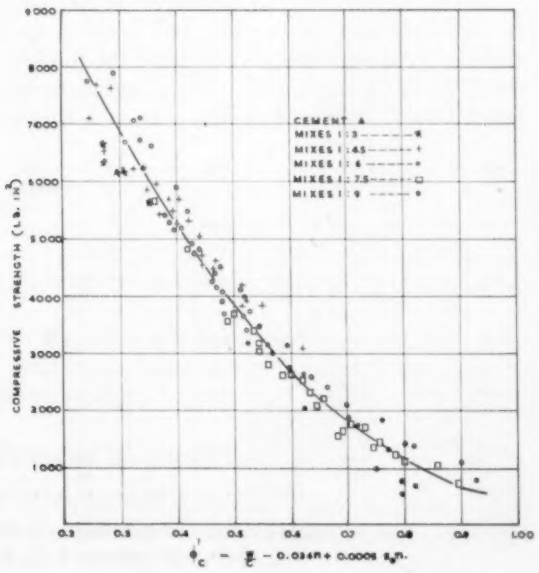

Fig. 7—ϕ_c^1 plotted against strength at 28 days for water-cured concrete of various mix proportions and aggregate of various specific surface

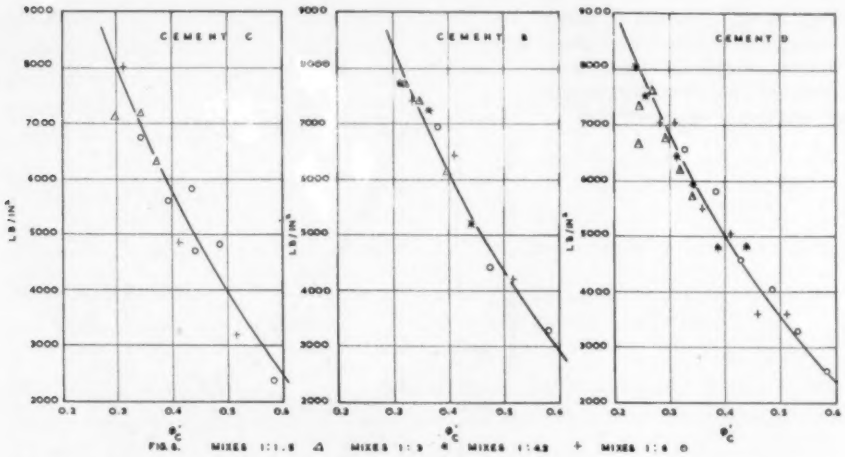

Fig. 8—$\phi_c{}^1$ plotted against compressive strength at 7 days for water-cured concrete made with cements B, C, and D

Considering the various mix proportions for cements *A*, *B*, *C*, and *D*, *K* was found to have a value of 0.034. Fig. 6, 7, 8, and 9 show that when $\phi_c{}^1$ is plotted against strength, continuous curves are obtained for the various gradings, mix proportions, and cements.

EFFECT OF SPECIFIC SURFACE ON FLEXURAL STRENGTH

In Fig. 10 the modulus of rupture is plotted against the water-cement ratio for constant mix proportion and varying specific surface. As the specific surface of the aggregate is increased the modulus of rupture at a given water-cement ratio is decreased. The same characteristics are shown by the other mixes tested. It would appear that as in the case of the compressive strength

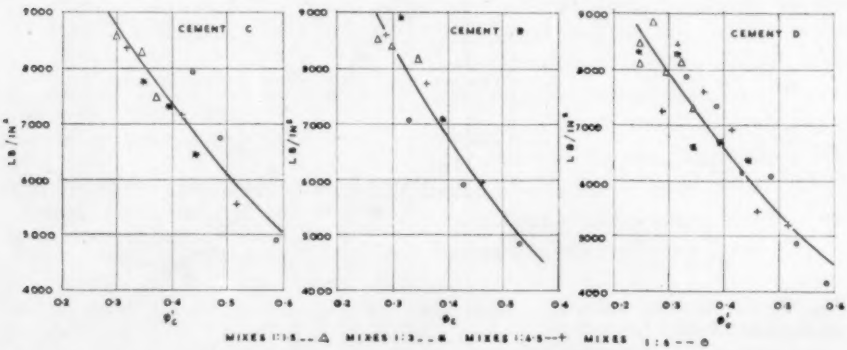

Fig. 9—$\phi_c{}^1$ plotted against compressive strength at 28 days for water-cured concrete made with cements B, C, and D

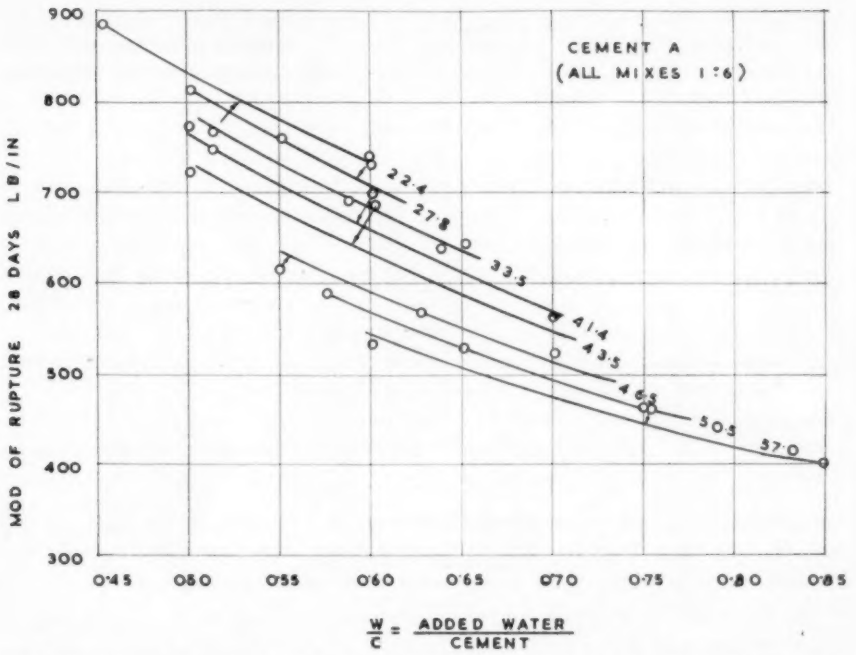

Fig. 10—Effect of grading on the modulus of rupture. As the specific surface is increased the modulus of rupture at a given water-cement ratio is decreased

the modulus of rupture decreases as the ratio of the surface of the aggregate to the cement available for cementing the aggregate particles is decreased. Considering the modulus of rupture as related to both the water-cement ratio and the area of aggregate per unit of cement, an equation similar to Eq. (3) could be considered, i.e.

$$\phi_r{}^1 = W/C - KN + K_r S_o N \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

In order to obtain a single curve (Fig. 11 and 12) for the various specific surfaces and mix proportions, K_r was found to have a value of 0.001, using $K = 0.034$.

The ratio of K_r is greater than K_c. This is a result that might be expected, for the specific surface of the aggregate should affect the tensile strength of concrete more than its compressive strength.

DISCUSSION OF RESULTS

It is clear from the results obtained that both the compressive strength and modulus of rupture of concrete are affected by the specific surface of the aggregate. However, the constants K_c, K_r, and K found in the present in-

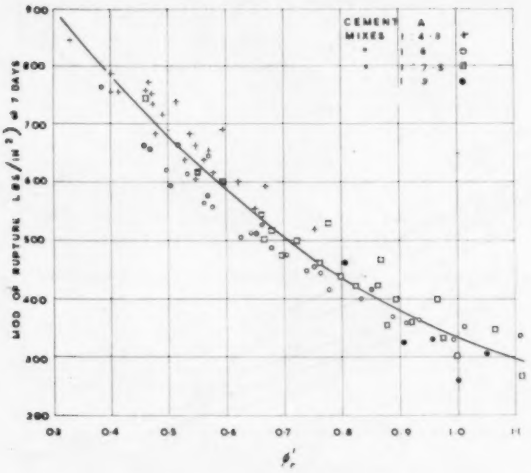

Fig. 11—$\phi_r{}^1$ plotted against modulus of rupture at 7 days for water-cured concretes

vestigation may not have general application. In fact, it is expected K_c and K_r will vary with the physical properties of the aggregate, though the physical interpretation of these constants is somewhat speculative.

The allowance of 3.40 percent for absorption, according to the interpretation given is high and seems somewhat dubious, as the normal absorption of the combined aggregate did not exceed even half this value. A more complete explanation for this high allowance might include a differential stress distribution within the concretes of different cement contents, as a result of the difference in elastic modulus of the aggregate relative to that of the cement matrix. It is conceivable that closer contact of the aggregate particles (which

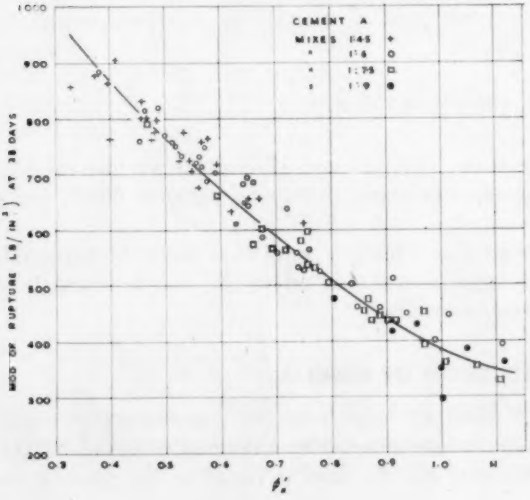

Fig. 12—$\phi_r{}^1$ plotted against modulus of rupture at 28 days for water-cured concretes

have a higher elastic modulus) in leaner mixes, for the same amount of water associated with the cement, might give a higher compressive strength. Any allowance made for this additional factor might also be proportional to N.

If the above explanation holds, then it is conceivable that the adjustment for "absorption" will not only vary with the type of aggregate but also for different cements and age of test. There is some evidence in the present investigation that K varies with time. For instance, for the K value of 0.034, more differential scatter is shown for mixes 1:3 and 1:9 in Fig 7, giving the 28-day test results than in Fig. 6 giving the 7-day results. Further, if this explanation holds, it indicates that a high strength concrete need not necessarily be a rich concrete.

There are undoubtedly other factors affecting strength in the present investigation apart from those mentioned. For example, Grading No. 9 (See Table 1) gave a higher strength consistently. The reason for this is not quite clear, though it might be taken as an indication of possible advantages to be obtained from certain gap-gradings. Though Grading No. 1 gave comparable compressive strength, it tended to consistently give low transverse strength results.

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REFERENCES

1. Abrams, D. A., "Design of Concrete Mixes," *Bulletin* No. 1, Structural Materials Research Laboratory, Lewis Institute, 1918 (revised edition 1925).

2. Weymouth, C. A. G., "Effect of Particle Interference in Mortars and Concrete," *Rock Products*, V. 36, No. 2, 1933, pp. 26-30.

3. Loudon, A. G., "A Laboratory Study of the Permeability of Sand," *Geotechnique* (London), V. 3, No. 4, pp. 165-183.

4. Newman, A. J., and Teychenné, D. C., *Paper* No. 10, Symposium on Mix Design and Quality Control of Concrete, London, 1954.

Discussion of this paper should reach ACI headquarters in triplicate by July 1, 1958, for publication in the Part 2, December 1958 JOURNAL.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Construction

Skyscraper of the Braunschweig School of Technology (Das Hochhaus der Technischen Hochschule Braunschweig)

HEINZ WREDE, *Der Bauingenieur* (Berlin), V. 32, No. 1, Jan. 1957, pp. 1-6

Reviewed by ARON L. MIRSKY

New structure, made necessary both by war damage to original buildings and by increased enrollment, is a tall (17 stories plus two basement floors), thin, long building (15 x 48 m in plan below ground, 10.1 x 41.5 m above ground) of reinforced concrete with precast concrete facing panels. Two-story basement was designed as hollow foundation slab, due to soil conditions; upper basement level is used for utilities, etc., while lower level affords space for an air-raid shelter.

Construction Technique

Practice of wood formwork and reinforcement. V. 3—Forming slabs, capitals, bridges, reservoirs, and precast elements (Pratique du coffrage en bois et du ferraillage V. 3—Le coffrage des planchers-champignons, des silos, des ponts, des réservoirs et des éléments coulés d'avance)

C. KUPFER, Editions Eyrolles, Paris, 1957, 94 pp., 940 F.

This third volume completes the series on this subject. The first two volumes treated general applications while the present one gives details on special problems in forming column capitals, silos, arch and slab bridges, reservoirs, and precast elements. This volume, like the earlier ones, is intended to permit more exact form designs and cost estimates.

Tips on tilt-up construction

SAMUEL HOBBS and F. THOMAS COLLINS, *Consulting Engineer*, V. 9, No. 10, Oct. 1957, pp. 80-85

Reviewed by ARON L. MIRSKY

Practical pointers on good design, with particular emphasis on pickup and jointing details. Besides a full handbook-type page of recommended joints, article includes two pages on two recent industrial buildings of tilt-up construction designed by Collins.

Effect of method of curing on internal temperatures of concrete pavements in North Carolina

C. E. PROUDLEY and A. D. MORGAN, *Proceedings*, Highway Research Board, V. 36, 1957, pp. 347-356; discussion pp. 356-358

AUTHORS' SUMMARY

Relative merits of four curing methods not involving the use of applied water were studied: (a) white pigmented liquid membrane, (b) clear liquid membrane with fugitive red dye, (c) tan waterproof paper, and (d) white waterproof paper. Moisture retention was not measured. Variations in temperature at three depths in the pavement slab were recorded from the time of placement until 72 hr.

Considering the temperature variables, the writers conclude that lowest temperatures are obtained with white waterproof paper, followed in order by white pigmented liquid membrane, tan waterproof paper, and clear liquid membrane with fugitive red dye.

Design

Statically-indeterminate structures

R. GARTNER, Concrete Publications, Ltd., London, 3rd edition, 1958, 130 pp., $4

The third edition includes an added chapter on design by the "plastic hinge method" for

A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 10, Apr. 1958, *Proceedings* V. 54. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

both structural steel and reinforced concrete. The text presents classical methods of structural analysis, moment distribution and specialized problems such as bow girders, Vierendeel trusses and complex building frames. It should be valuable as a specialized reference to practicing structural engineers or teachers of advanced structural analysis courses.

Design of eccentrically-loaded columns by the load-factor method. Part 2—Symmetrical mild steel reinforcement

J. D. Bennett, *Concrete and Constructional Engineering* (London), V. 52, No. 12, Dec. 1957, pp. 411-416

Contains five design charts for columns symmetrically reinforced with mild steel. Design equations are given and results compared with British Standard Code of Practice No. 114 (1957). Part 1 on columns reinforced with cold worked bars was reviewed in "Current Reviews," ACI Journal, Feb. 1958, p. 712.

Model analysis of Belfast power station building frame

Kim-Fung Tse, *Civil Engineering and Public Works Review* (London), V. 52, No. 612, June 1957, Part 1, pp. 660-662

Presents an application of the method of analyzing structures using large models previously developed by the same author. Concludes that the usual assumption of negligible effects of axial strains may in some cases render the solutions achieved completely useless.

Edge stresses in thin shells of revolution

W. H. Wittrick, *Australian Journal of Applied Science* (Melbourne), V. 8, No. 7, Dec. 1957, pp. 235-260

Author's Summary

Explicit solutions are obtained for stress systems which decay rapidly with the distance from either a circular or a meridional edge of a thin shell whose shape is a surface of revolution, but otherwise arbitrary. The accuracy of the solutions is consistent with that of the generally accepted equations of thin shell theory. The significance of these solutions from the point of view of the boundary conditions of membrane theory is discussed.

Bending of semi-circular thin plates with restrained circular edge and free diameter edge (in Russian)

G. A. Grinberg and R. P. Poplavski, *Inzhenernyi Sbornik*, Akademii Nauk SSSR (Moscow), No. 18, 1954, pp. 83-88

Reviewed by J. J. Polivka

Thorough analysis of a thin semi-circular plate for any type of loading on the basis of elastic deflections expressed in polar coordinates. Method is demonstrated by numerical example, assuming uniformly distributed load. It is also explained how this method can be applied to this structural type under other conditions, e.g. similar plate freely supported around the circular boundary. Author refers also to paper discussing the method of analyzing rectangular thin plates originated by P. F. Papkovich.

Axisymmetric deformation of cylindrical shells in elastic and plastic deformation (in Russian)

A. N. Ananina, *Inzhenernyi Sbornik*, Akademii Nauk SSSR (Moscow), No. 18, 1954, pp. 157-160

Reviewed by J. J. Polivka

Approximate solution presented is based on the assumption that the elastic stage is transformed in plastic state over the whole thickness of the shell, with the limitation that the yield point is not reached in any fiber. Comparison with exact solution by A. A. Iliushin justifies this approximate solution.

Shear, diagonal tension, and bond stresses in reinforced concrete beams

E. M. Rensaa, reprinted from *The Engineering Journal* (Montreal), Oct. 1957, 8 pp.

Discusses problems in design of reinforced concrete which have recently arisen in connection with several failures of reinforced concrete building frames. The problems particularly include shear conditions at points of contraflexure in a rigid frame as well as the relationship between bond, anchorage, and splitting of concrete. In particular the author discusses inadequacies in present practice, both Canadian and United States, in design for conditions near the points of contraflexure. Suggested strengthening of code provisions for these cases is well supported by his arguments. In any event the paper should be of interest to all practicing structural engineers and in particular those on committees preparing codes.

Buckling of unreinforced concrete walls (in Norwegian)

R. Schjodt, *Nordisk Betong* (Stockholm), No. 4, 1957, pp. 305-310

Reviewed by Margaret Corbin

The load-carrying capacity of plain concrete walls is limited to a large extent by unforeseen bending moments. When these exceed a certain limit cracks will develop at top or bottom, forming a plastic hinge and setting an upper limit for the magnitude of the moments. The above is true when the resisting moment is at top or bottom; an eccentric load may result in a maximum moment nearer the center of the wall. This condition is used as a criterion of buckling. Computations show that a plain concrete column fixed at the bottom may be assumed to have an equivalent length of 1.2 times the length of a similar column of a homogeneous material. If the column is hinged at the bottom, the equivalent length is 2.0 times length of similar column of homogeneous material. The author proposes use of an equivalent length of 1.6 times.

Effect of the radial stress on the resisting moment in plastic hinge

I. Ohno, *Proceedings*, 6th Japan National Congress for Applied Mechanics (Tokyo), 1956, pp. 201-204

Presents the theoretical solution for a plastic hinge in a curved beam in which the effect of the radial stress in addition to the normal stress must be considered. In addition considers the effect of the shearing stress. Numerical examples are given. Concludes that the effect of the radial stress is to decrease somewhat the bending moment in the plastic hinge, and that this effect increases as the radius of curvature of the axis of the curved beam decreases.

Myklestad's method for nonuniform vibrating beams

R. E. D. Bishop, *The Engineer* (London), V. 202, 1956: No. 5264, Dec. 14, pp. 838-840; No. 5265, Dec. 21, pp. 874-875

Reviewed by Aron L. Mirsky

Presents tabular method for calculation of receptances of beams of nonuniform section. Tables developed by author are simpler than those previously used. Method is explained by worked examples; accuracy of calculations was checked experimentally.

On the calculation formulae of differential settlement of structures

Y. Yakoo and K. Yamagata, *Proceedings*, 6th Japan National Congress for Applied Mechanics (Tokyo), 1956, pp. 161-164

A continuation of previous papers developing theoretical solutions based on theory of elasticity for the effect on structures of differential settlements. Two methods are shown, one in this paper in which final settlement of column points are the unknowns and one from previous papers in which the column point base reactions are unknowns.

New solution and calculation of arch loaded with a torsional moment

S. Nakamura, *Proceedings*, 6th Japan National Congress for Applied Mechanics (Tokyo), 1956, pp. 171-174

Presents an elastic solution for the effect of torsional moments on an arch which is hinged against moments in its axial plane and fixed against twisting moment at both ends. The solution is also developed for a hingeless arch.

Analytical and physical data on the flexibility of beams on elastic ground (in Italian)

Raffaello Rosardo, *Giornale del Genio Civile* (Rome), V. 95, No. 2, Feb. 1957, pp. 115-120

Reviewed by John I. Prelez

The author gives a particular physical interpretation to the flexibility of beams which enabled him to get satisfactory results especially for road and pavement design.

Use of high-tensile steel reinforcement

K. Hajnal-Konyi, *Concrete and Constructional Engineering* (London), V. 52, No. 12, Dec. 1957, pp. 397-402

A general discussion of conventional versus ultimate strength design methods. As a result of charts comparing designs by both methods, the conclusions reached are (1) that the economical use of high strength concrete necessitates the use of high-tensile steel and (2) that the ultimate load method of design as introduced in the new British Code has eliminated the handicap under which the use of high-tensile steel has suffered on the basis of the standard method. Twenty-five references include recent ACI and European papers.

Strain energy methods for the solution of deep beams

F. E. Archer and E. M. Kitchen, *Civil Engineering and Public Works Review* (London), V. 52, No. 618, Dec. 1957, pp. 1375-1378

Presents a complete derivation for the strain energy method applied to the solution of the deep beam problem. Compares results by this method with the simple beam theory and results by photoelastic studies for both longitudinal stresses and shear. Concludes that the strain energy method gives results which are quite compatible with those by the method of photoelastic analysis.

Materials

Sydney fly ash in concrete—1

G. B. Welch and J. R. Burton, *Commonwealth Engineer* (Melbourne), V. 45, No. 5, Dec. 1957, pp. 48-53

Describes research on the effect of Australian fly ash used as an artificial pozzolan. The fly ash available was "inferior in some ways to better class ashes in use in the U. S. A.," the principal difference being higher carbon contents and less fineness. Results of the investigation were encouraging to use of Sydney fly ash, particularly as an admixture or as a sand replacement, but as a cement replacement only for a very low percentage of cement.

Chemicals and concrete

M. J. Snyder and N. G. Foss, *Modern Concrete*, V. 21, No. 7, Nov. 1957, pp. 41-44, 58

Summarizes a complete survey by Battelle Memorial Institute of major chemicals used with concrete. The survey included chemicals used in concrete as admixtures and on concrete as curing compounds and surface treatments. Concludes with a review of the market potentials for these chemicals.

A reactive aggregate undetected by ASTM tests

E. G. Swenson, *ASTM Bulletin*, No. 226, Dec. 1957, pp. 48-51

A progress report on a Canadian investigation of a dolomitic limestone coarse aggregate which has been found to produce excessive expansion and cracking in concrete. The expansion is quite evidently caused by some sort of alkali-aggregate reaction but the standard tests on the aggregate gave negative results for alkali reactivity. The unsatisfactory field performance of the aggregate in question has been duplicated in concrete test prisms under various exposures. The concrete prism tests indicate that only the high alkali cements produce excessive expansion and that the usual remedies of pozzolanic or chemical admixtures are not particularly effective with this cement-aggregate reaction.

Tests of concrete containing portland blast-furnace slag cement

William E. Grieb and George Werner, *Proceedings*, Highway Research Board, V. 36, 1957, pp. 253-265

Authors' Summary

The results of the strength tests indicate that portland blast-furnace slag cement develops strength more slowly than portland cement, but at an age of 90 days will furnish higher strength concrete. No marked difference has been found in the shrinkage of concretes prepared with the two kinds of cement. The freezing and thawing tests have not progressed sufficiently to indicate trends applicable to the cements used.

The tests for strength of concrete will be continued to age of 1 year. Freezing and thawing tests will be terminated at 300 cycles. The drying shrinkage tests will be continued for a total of 180 days.

Concrete reinforced with glass

Civil Engineering and Public Works Review (London), V. 52, No. 615, Sept. 1957, p. 996

Highway Research Abstracts Jan. 1958

The Russian periodical *Stroitelnaya Promishlennost*, No. 6, 1957, contains some interesting notes on the use of glass for concrete reinforcement. The investigations have been carried out by the Kiev polytechnic structural laboratory. The glass was of the aluminum and boro-silicate composition and the strands, which had a diameter of 5-6 microns, had the breaking strength of 330,000-360,000 psi with the modulus of elasticity $E = 10.7 \times 10^6$ psi.

The alkaline resistance of glass strands was tested and some disintegration took place. The use of larger diameter strands is advised (20 microns diameter minimum) if no protective coating is applied. The strands, coated with synthetic resins like phenol formaldehyde, aniline formaldehyde, and

polivinyl chlorides, showed no signs of deterioration after 6, 12, and 30 months. The bond stress varied from 430 to 1,100 psi (30 to 75 kg per sq cm), coated strands showing a better bond. Two groups of tests were made with the glass-concrete; one with untensioned and another tensioned strands.

Pavements

Pumping of highway and airfield pavements

E. J. YODER, *Proceedings*, Highway Research Board, V. 36, 1957, pp. 388-426; discussion pp. 426-432

AUTHOR'S SUMMARY

This paper presents the results of a study of pumping of rigid pavements. A literature search was conducted on present and past experiences of state highway departments and various governmental agencies relative to the pumping problem.

A study was made of the mechanics of pumping of rigid pavements. Surveys were made of highway pavements in Indiana constructed with and without granular bases, as well as of 15 airfields.

The report is divided into three parts: (a) a review of pumping of highway pavements built directly on natural subgrades, (b) performance of rigid highway pavements built on granular bases, and (c) a study of pumping of airfield aprons, taxiways, and runways.

Load transfer characteristics of a dowelled joint subjected to aircraft wheel loads

J. R. KEETON and J. A. BISHOP, *Proceedings*, Highway Research Board, V. 36, 1957, pp. 190-198; discussion, pp. 199-206

AUTHORS' SUMMARY

Interim results of a study being conducted on the load transfer characteristics of dowels used in airfield pavement expansion joints are presented. The slab consists of two 10 in. thick, 15 ft wide, and 25 ft long sections joined by a ¾-in. expansion joint across which 1⅛ in. diameter dowels are placed on 12-in. centers. Each dowel is instrumented with strain gages to measure bending moment along the dowel and shear transmitted across the joint. Moment, shear, and pressure curves are presented for all dowels in the joint for a wheel load of 50,000 lb applied at the center of the slab tangent to one face of the joint. Slab deflections are measured with mechanical dial indicators and complete transverse and longitudinal contours are presented. Results of the application of test results to current design equations are presented.

Heavy wheel load traffic on concrete airfield pavements

FRANK M. MELLINGER, JAMES P. SALE, and THURMAN R. WATHEN, *Proceedings*, Highway Research Board, V. 36, 1957, pp. 175-189

AUTHORS' SUMMARY

Testing described deals chiefly with the two factors which greatly influence the design of an airfield pavement, namely, the magnitude of the wheel loadings and the number of repetitions of these loadings during the design life of the pavement. Two full-scale pavement test sections, one on a uniform subgrade of high bearing value, the other on a uniform subgrade of low bearing value, are described, and the results of accelerated traffic testing are presented and discussed. Each of the sections contains comparable designs of plain and reinforced concrete. Each test section was subjected to 90,000 repetitions of a 100,000-lb twin-wheel loading.

Performance of an experimental project to determine the efficiency of several plain and reinforced concrete pavements

CARL E. VOGELGESANG and WILMER E. TESKE, *Bulletin* No. 165, Highway Research Board, 1957, pp. 1-34

Describes the design and performance of an experimental concrete pavement constructed in 1931 a few miles north of Indianapolis on State Road 29. Purpose was to determine the relative efficiency of several designs of reinforced concrete pavement, expansion joints, and contraction or dummy joints. Nine special design sections, each approximately 2700 ft long, and seven standard sections were incorporated in 8.35 miles of test pavement. Special design features in the nine sections included variations in the type and amount of reinforcement, type and spacing of transverse joints, and type of joint filler.

In general, wire mesh reinforcement was found more effective in preserving the continuity of the pavement than the bar mat type of reinforcement. Based on surface

roughness measurements made in 1941 and 1954 some reinforced sections have improved in surface smoothness during the past 13 years, while all standard sections have shown a decided increase in roughness.

Thermal effects in concrete highways (Les effets thermiques dans les chaussées en béton)

G. JEUFFROY, *Annales des Ponts et Chaussées* (Paris), V. 127, No. 2, Mar.-Apr. 1957, pp. 227-256
Reviewed by ARON L. MIRSKY

Starting with a study of surface temperatures (assumed to vary sinusoidally with time) and temperature distribution in a slab (assumed to obey the Fourier theory), author develops curves of thermal stresses in slabs in terms of thickness and size of slab and daily surface temperature variations. Thermal shocks, due to sudden variations of the surface temperature, are discussed briefly. Author concludes that in general it is good practice to keep slab dimensions fairly small, with 5 m (16.4 ft) being the limit for continental European climatic conditions.

Methods of stabilization of earth and granular material, and suggestions for design of flexible pavements (in Italian)

VINCENZO VITTORIA, *Giornale del Genio Civile* (Rome), V. 95, No. 4-5, Apr.-May 1957, pp. 336-342
Reviewed by JOHN I. PRELEZ

Discusses briefly some fundamental notions regarding the techniques of stabilization of roads, and reviews the most common methods used in designing flexible pavements. The author applied these principles and methods during highway construction in Argentina.

Test project constructed utilizing the contraction joint design

ERNEST T. PERKINS, *Bulletin* No. 165, Highway Research Board, 1957, pp. 35-46

A slab design was developed to test the performance of contraction joints and to confine the joint opening to controllable limits within which known sealers would remain effective.

In 1952, a section of concrete pavement, approximately 1800 ft long between expansion joints, was constructed on the Wilbur Cross Highway in Vernon, Conn. Intermediate contraction joints with load transfer provision were installed at intervals of 37 ft 4 in., six different types of load transfer assemblies, in groups of eight, being used. Observations on its performance are reported.

Precast Concrete

Contribution to solve the continuity problems occurring in floor slabs and beams of precast reinforced concrete (in Rumanian)

G. KLOCKNER and C. RADULESCU, *Industria Constructiilor si a Materialelor de Constructii* (Bucharest), V. 8, No. 9, 1957, pp. 496-501
Reviewed by J. J. POLIVKA

Beams are precast in V-shape with transverse concrete stiffenings 4 ft on centers. Two types of interconnecting slabs are described. Type U_1 are precast slabs, 1 in. thick, with transverse joints, to be filled with concrete for continuity. Continuity with beams is provided by concrete cast in spaces of V-shape beams, and additional reinforcement. Type U_2 is cast in place on arched steel sheets, and the thickness of the slab increases from 1¼ to 1¾ in. Great economy of this structural type is discussed.

Prefabrication plant for members in reinforced concrete "Vibrobeton" in Sophia (in Rumanian)

B. MARACINE, *Industria Constructiilor si a Materialelor de Constructii* (Bucharest), V. 8, No. 9, 1957, pp. 509-519
Reviewed by J. J. POLIVKA

Thorough description of a large factory with all mechanical equipment for producing different types of structural members, pipes, piles, etc. Methods of fabrication, their economy, and statistics concerning efficiency and production are given.

Precast prestressed concrete jetty, Inverkeithing, Scotland

British Constructional Engineer (London), V. 8, No. 3, Nov. 1957, pp. 31-32
Reviewed by K. R. LAUER

The working platform spans over a steep slope of rock by means of precast prestressed beams resting on a capping beam formed over piles. The prestressed beams 2 ft 6 in. on center vary in length from 40 to 50 ft. Each consisted of a site precast rib, 3 ft 2 in. high with flange widths of 10 in. and a web thickness of 6 in. The beams were post-tensioned in two stages on the P.S.C. one-

wire system. Prestress was applied in the first stage immediately after the concrete test cubes showed a minimum crushing strength of 5000 psi.

A concrete mix of 1:1:2 by weight was used with a water-cement ratio of 0.35. The aggregate was crushed granite of ¾ in. maximum size. Compressive strengths averaged 5000 psi at 3 days and consistently exceeded 9000 psi at 28 days.

Tetrapods protect island shore line

R. S. Torgerson, *Concrete Products*, V. 61, No. 1, Jan. 1958, p. 34

Pictures and describes production of 1100 31-ton tetrapods in a California products plant for shore protection.

Precast concrete facing elements

G. R. Saunders, *Prefabrication* (London), V. 5, No. 51, Jan. 1958, pp. 120-125

Pictures architectural details employed in the use of precast concrete wall panels. Includes complete description of the various surface finishes available and description and examples of their use as (1) permanent formwork, (2) slabs fixed to wall, (3) slabs standing horizontally from column to column with cavities, (4) slabs fixed vertically.

They precast concrete docks that float

Concrete, V. 66, No. 2, Feb. 1958, pp. 32-33

Precast concrete residential floating dock units vary in size from 3 x 8 to 6 x 8 ft, and several may be attached to secure a dock of almost any size and shape. The units are hollow boxes composed of lightweight aggregate (expanded shale) concrete.

Simplified method for erecting heavy precast arches in reinforced concrete (to be used for arch bridges) (in Rumanian)

Oreste Vorobcius, *Industria Constructiilor si a Materialelor de Constructii* (Bucharest), V. 8, No. 9, 1957, pp. 483-495

Reviewed by J. J. Polivka

Various methods of erecting precast and prestressed arched members are described and illustrated, to be used especially for hinged arches, shell-type bridges (Maillart system), and arch bridges with suspended deck structure. Methods are demonstrated by actual examples of three-hinge arches spanning 164 ft. Structural elements weighing up to 100 tons were erected with typical cranes and trolley-mounted wood-frame towers. Additional reinforcing is to be provided for temporary stresses produced during erection.

Prestressed Concrete

Contribution to the analysis of prestressed-concrete structures (Beitrag zur Berechnung von Spannbeton-Konstruktionen)

Konrad Sattler, *Der Bauingenieur* (Berlin), V. 31, No. 12, Dec. 1956, pp. 444-457

Reviewed by Aron L. Mirsky

Author develops unified method of attack for prestressed concrete structures, applicable to both statically determinate and statically indeterminate systems and including the effects of both shrinkage and creep. For the statically indeterminate systems, he develops a number k whose magnitude determines whether the moment of inertia of the steel (prestressing tendons plus any untensioned mild steel bars) about its own center of gravity axis must be included in the analysis. Author concludes amount of untensioned steel should be kept as small as possible, contrary to German Standard DIN 4227 which specifies a minimum of 0.3 percent of the concrete section. Several numerical examples are included.

Reviewer would have found this paper much more interesting and informative had the symbols, of which there are a goodly number, been defined. Some are of course obvious; others will no doubt be familiar to European readers, but the casual reader or even the researcher may find himself somewhat lost.

Endurance of a full-scale pretensioned concrete beam

K. E. Knudsen and W. J. Eney, *Proceedings*, Highway Research Board, V. 36, 1957, pp. 103-128

Authors' Summary

Behavior of a strand-reinforced pretensioned concrete beam was observed under 1,300,000 repetitions of an equivalent standard H20-S16 (36-ton gross weight) truck loading, for which it was designed, and a subsequent 100,000 repetitions of 54 percent design overload. The beam was 38 ft long, 36 in. wide, and 21 in. deep, and contained 40 5/16 in. diameter 7-wire strands, with a

total steel area per beam of 2.25 sq in. Seven static tests, each with loads ranging from 0 to 100 percent of design load, and six more with 0 to 410 percent of design load, were made concurrent with the repetitive load tests.

The beam proved to be structually sound and entirely satisfactory for its use as a highway bridge member for a lifetime of service in a most severe location. The tests confirmed the adequacy of the methods of analysis to predict beam behavior.

Exact curvature of pretensioning cable (in Danish)

J. F. Bohmer, *Tekniska Skrifter* (Stockholm), No. 14, 1956, 11 pp.

Applied Mechanics Reviews Jan. 1958

Author deals with the problem of determining the exact curvature of the pretensioning cable which, owing to uniform distributed load, will give momentless stress distribution throughout the beam.

Prestressed concrete column under eccentric loading

T. Y. Lin and R. Itaya, *Journal of the Prestressed Concrete Institute*, V. 2. No. 3, Dec. 1957, pp. 5-17

Describes the test and analysis of a pretensioned 16-in. octagonal concrete column 25 ft long subjected to an eccentric load. Elastic and plastic behavior of the column were determined both experimentally and analytically. Two analytical methods of analysis for ultimate load capacity are proposed. It is concluded that more experimental data are required to establish a reliable method of analysis.

Prestressed concrete water towers (Réservoirs en béton précontraint)

J. M. Deharveng, *La Technique Modern-Construction* (Paris), V. 12, No. 9, Sept. 1957, pp. 276-279

Reviewed by Alexander M. Turitzin

Describes prestressed concrete water towers of cylindrical and conical shapes designed and built in France from 1953 to 1956. Many photographs, sketches, and details showing location of reinforcing steel and prestressing strands are given. The capacity of water towers varied from 500 to 2000 cu m. Of interest is a table of materials for nine different projects, giving the total quantity, and the quantity per cu m of storage capacity, of concrete, prestressing strands, and conventional reinforcing. The author believes that the main reason for using prestressed concrete in water tower construction is that it makes tower walls watertight. Prestressing keeps concrete walls under permanent state of compression, thus eliminating the possibility of shrinkage and temperature cracks.

Composite prestressed concrete floor

K. K. Banerjee, *Journal of Technology*, Bengal Engineering College (Calcutta), V. 2, No. 1, June 1957, pp. 23-48

Discusses advantages and problems of composite prestressed concrete floor construction. Develops design procedures for this type of floor, including the effect of differential shrinkage with an example. Predicted results are compared to full-scale tests. Concludes that substantial economies may be achieved through composite construction. Includes nine recent references to British and German literature.

Proceedings of the World Conference on Prestressed Concrete

World Conference on Prestressed Concrete, Inc., 417 Market St., San Francisco, 600 pp., $10

Part 1 includes the 38 papers orally presented at the conference in San Francisco, July 1957. Part 2 includes an additional 26 papers (not orally presented) which were distributed at the conference. Authors and speakers included authorities from all parts of the world. General subjects include: (1) materials and techniques; (2) prestressed bridges; (3) prestressed buildings; (4) prestressed pavements, floors, piles, dams, and masts; (5) research on prestressed concrete; (6) manufacture of precast prestressed concrete; (7) design and construction in various countries.

Individual papers describe construction of noteworthy prestressed concrete structures of all types in all parts of the world including slab bridges, girder bridges, trestle bridges, prestressed concrete suspension bridges, prestressed lift slabs, prestressed and precast shells, prestressed pavement, piles, transmission line poles, folded plates, and tension dome roofs. Research papers contributed data on various systems of prestressing, effect of admixtures, shear strength, flexural strength, torsional strength, loss of prestress due to friction, analysis of stresses in end

blocks, use of expanding cement, and fire resistance of prestressed concrete.

Included is complete registration list of the 1200 persons who attended the convention. Durably and attractively bound, the book should form a valuable permanent addition to the reference library of practicing engineers.

High structural performance: advanced applications of prestressing

A. J. Harris, *Engineering* (London), V. 184, No. 4772, Aug. 23, 1957, pp. 244-246

Reviewed by Aron L. Mirsky

Basic concepts of prestressing are briefly but convincingly presented, and its advantages are demonstrated; the technique is shown to be applicable not only to concrete (to which it has up to now been largely restricted) but to other materials as well, in cases where high structural performance is needed.

Properties of Concrete

Concrete and cement in sea water

W. Kronsbein, *Zement-Kalk-Gips* (Wiesbaden), V. 10, No. 7, July 1957, pp. 288-291

Ceramic Abstracts Feb. 1978 (Hartenheim)

Tests on concrete with a cement content of 300 kg per cu m and a gravel-sand aggregate with a grading between the standard curves D and E showed marked differences in water permeability depending on the quality of the cement used. The addition of trass, because of its swelling properties, effectively improved the poor water retention and workability (found especially when blast-furnace cement is used) which are a major cause of unsatisfactory water permeability of concrete. Trass also makes up the deficiency of fines that exists in many aggregates.

Uncertainty in determining concrete properties (in Swedish)

Birger Warris, *Nordisk Betong* (Stockholm), No. 4, 1957, pp. 311-320

Reviewed by Margaret Corbin

Paper deals with the definition, significance, and magnitude of the uncertainty met in determining the properties of concrete on the basis of laboratory measurements. This uncertainty is expressed in terms of the range or the standard deviation. The *t*-test, among other things, is used to illustrate the importance of the uncertainty. Examples show how knowledge of uncertainty can be utilized in the planning of tests and for the elimination of extreme values observed in tests. These examples deal with measurements of air content in hardened concrete by means of a linear traverse and consistency measurements. Uncertainties are classified as follows: standard deviation within a batch; standard deviation in different batches placed immediately after each other; standard deviation within a laboratory; standard deviation in values observed in different laboratories.

Some considerations of the cracking or fracture of concrete

F. A. Blakey, *Civil Engineering and Public Works Review* (London), V. 52, No. 615, Sept. 1957, pp. 1000-1003

Highway Research Abstracts Jan. 1958

Present knowledge of the conditions governing fracture of concrete has been summarized. Cracking of concrete may start by: (a) fracture of cement paste, (b) fracture of aggregate, or (c) failure of bond between paste and aggregate.

By an idealization of the problem and a simple method of analysis it may be possible to predict which origin of failure is most likely when the properties of the materials are known.

No account has previously been taken of the heterogeneity of concrete and no experimental work is known in which the stress distribution between the different phases in the concrete has been measured. The results can be tested by observation and the validity of the original assumptions assessed.

Structural Research

Small-scale load tests on drilled and cast-in-place concrete piles

Lawrence A. DuBose, *Proceedings*, Highway Research Board, V. 36, 1957, pp. 132-145

Author's Summary

As a portion of a more extensive study on drilled and cast-in-place concrete piles, special tests were conducted on small-scale piles which had been placed in a 30-in. cube of laboratory compacted soil. One series of small-scale piles was used to observe the

influence of several factors on water movements from unset concrete into an annular zone of soil immediately surrounding the piles. These tests indicated that water movements were negligible, except when the initial water content of the compacted soil (for the test cubes) was several percent lower than the plastic limit of the soil. Both tension and compression load tests were conducted on small piles and the tension capacity of the piles was found to be between 0.6 and 0.7 of the compression pile load capacity. Other special tests were made to observe the nature of soil deformations adjacent to the pile surface during a load test and the portion of the pile load which was resisted by end-bearing stresses at the base of the pile.

Instrumentation and method for impulse testing of reinforced concrete beams

F. T. Mavis and M. J. Greaves, *ASTM Bulletin*, No. 226, Dec. 1957, pp. 43-47

Describes the measurement apparatus which superimposes oscilloscope records and high speed motion pictures without time parallax in recording the data of impulse tests on pairs of reinforced concrete beams of comparable design strength, on the basis of present design practice in which hard grade steel was used in one beam and intermediate grade steel in the other. The results indicated that under impulse simulating blast loadings the ultimate strength of such beams was a function of the yield strength of the reinforcement. (Cf. "Destructive Impulse Loading of Reinforced Concrete Beams," by these authors, ACI Journal, Sept. 1957, *Proc.* V. 54, pp. 233-252.)

Spiral socket splices for deformed bars

Axel Ersen, *Bulletin* No. 8, Research Laboratory of Building Technique, Technical University of Denmark (Copenhagen), 1957, 13 pp.

Two parts of main bar to be spliced are placed butt ends together, and the joint is enclosed by four or five splice bars arranged in a circle inside a spiral made of plain bars, with splice bars and spiral welded together at the ends. Author considers spiral splice a suitable replacement for ordinary splices where concrete is the medium to carry stresses from rod to rod, because the design is symmetrical character and the play of forces takes place in confined part of concrete within the spiral.

Tests on concrete prisms and beams were conducted to determine the effect on maximum bond stress of variations in concrete quality, spiral reinforcement, surface deformations of main bars, and length of splice. The latter was shown to have no significant effect on maximum bond developed. Prism specimens had one spliced main bar at the axis; two bar ends protruded from the prism, and could be placed in the testing machine.

Charts and tables summarize results of four series of tests. Author recommends spiral socket length l no less than 20 to 25 times diameter of main bar; in final series of tests l was as much as 38 d, and in some cases that author favors still larger l. Some consideration is given to using decreasing splice length by using stronger concrete locally in the splice zones.

The prism pull-out test is suggested as a test for classifying the grip of various deformed bars.

Strain measurements under nonuniform vertical loading of self-supporting cylindrical shells (Mesures de déformation des voiles minces cylindriques autoportants sous des surcharges verticales non uniformément réparties)

J. Larras, *Travaux* (Paris), No. 268, Feb. 1957, pp. 71-74

As in the case of the earlier tests, these tests refer to self-supporting cylindrical shell roofs made of reinforced concrete with a length of about double the span; they are supported by rigid gable-ends without stiffeners or longitudinal beams.

The tests were for the purpose of studying the strains of a shell carrying a uniform load either over one-half of the profile or over the central part of the shell or in a more asymmetric position.

The tests have confirmed the flexible adaptability of self-supporting cylindrical shells to nonuniformly distributed vertical loads. Furthermore, they have shown that the maximum strain of a thin shell built in the usual way is between one-quarter and one-half of the thickness for loads of the order of the dead weight.

Fatigue tests on Lee-McCall prestressed concrete beams

W. Eastwood, *Civil Engineering and Public Works Review* (London), V. 52, No. 613, July 1957
Reviewed by Gunhard-Aestius Oravas

Describes fatigue tests carried out on some concrete beams prestressed by the Lee-McCall method.

Experimental research on light reinforced concrete beams (in Italian)

Tullio Renzulli, *Giornale del Genio Civile* (Rome), V. 95, No. 2, Feb. 1957, pp. 121-125
Reviewed by John I. Prelez

Reports stress data (up to the breaking point) of light reinforced concrete beams, under concentrated loads and three bearing points. Stress data obtained by laboratory method compared to those supplied by limit design are similar.

Testing post-tensioned slab and beam without grouting

Zachary Sherman, *Proceedings*, ASCE, V. 83, No. ST 4, July 1957, pp. 1317-1–1317-17
Author's Summary

Two industrial building post-tensioned members were tested. One was a 5 ft x 30 ft x 11 in. roof channel slab, and the other was a 25 ft long roof girder. The wire units were coated with mastic, and were not grouted in. The end anchorages are steel plate shims.

Load versus deflection curves were plotted, and cracking and ultimate loads were noted. Stress diagrams were computed and drawn. Factors of safety were also determined.

General

Rehabilitation of wharves and piers—army base, South Boston, Mass.

Journal, Boston Society of Civil Engineers, V. 44, No. 2, Apr. 1957

General	Robert J. Basso	pp. 70-74
Design	F. L. Lincoln	pp. 75-89
Construction	Joseph Peraino	pp. 90-97

Reviewed by Aron L. Mirsky

After considerable investigation and study, a cast-in-place gravity-type seawall,* with sloping front face and a cantilever deck, was selected. First paper describes the situation prior to rehabilitation; second paper discusses the various schemes studied, and the final design; last paper describes the construction, complicated by the necessity of keeping disruption of pier operations at a minimum.

*Story and pictures of formwork for this project appeared in the ACI "News Letter," Oct. 1956, pp. 11, 13, 17—Editor.

Gage for the determination of base friction in foundations (Messdose zum Ermitteln der Sohlreibung bei Fundamenten)

Hans-Heinrich Emschermann, *VDI Zeitschrift* (Düsseldorf), V. 99, No. 10, Apr. 1, 1957, pp. 423-425
Reviewed by Aron L. Mirsky

Vertical loads on rigid footings produce not only vertical distortions of the foundation material, but lateral distortions as well. These latter affect the footing via horizontal frictional forces acting on the bottom of the footing. Paper describes an ingenious device developed to measure these horizontal frictional forces accurately. Calibration of device is also described.

Conference on shielding of high-energy accelerators

Publication TID-7545, Technical Information Service Extension, Atomic Energy Commission, Dec. 1957, $5.50 (available from the Office of Technical Services, Department of Commerce, Washington 25, D. C.)

Proceedings of a conference sponsored by the U. S. Atomic Energy Commission Division of Research and the AEC New York Operations Office, Health and Safety Laboratory. The proceedings contains sections on shielding facilities of accelerators existing at the present time and a section on shielding design for accelerators now under construction. Another section covers shielding computations.

This is a good review of the shielding facilities of existing accelerator installations and their associated problems that can be used as a guide by those who will be involved in the design of shielding for new machines or the design of shielding modifications of existing accelerators.

Buildings for industry

Compiled from *Architectural Record*, F. W. Dodge Corp., New York, 308 pp. $9.75

Presents recent solutions to design problems for buildings in the following fields: warehouses, light industry, consumer goods, manufacturing laboratories, utilities and service industries, and heavy industry. Particularly well illustrated by over 700

pictures. Brief analyses of the design problems encountered in the 74 projects are also offered. The book should be a good source of new ideas as well as practical design data for architects.

1958 concrete products yearbook (Betonstein Jahrbuch 1958)

Bauverlag GMBH, Wiesbaden, 1958, 430 pp., 6.50 DM

Latest revision of the standards yearbook for concrete products, precast concrete, concrete masonry, and associated fields.

New method of recording vibrations in the foundation of turbogenerators and the structure of power houses

A. Major, *Acta Technica Academiae Scientiarum Hungaricae* (Budapest), V. 18, No. 3–4, 1957, pp. 295-316

Describes a method and results of measurements by the method applied to existing turbogenerator foundations. The method should be particularly valuable when it is desired to modify existing designs for improved characteristics or to supply data for guidance in new design.

Geoelectric measurements on concrete work (Geoelektrische Messungen in de Betonbautechnik)

Volker Fritsch, *Zement und Beton* (Vienna), V. 1, No. 7, Nov. 1956, pp. 11-16

Reviewed by Fritz Kramrisch

Methods employing electric potential fields, similar to those used in the exploration of soil formations, have been applied to the examination of concrete structures and their subsoil conditions.

The article describes the fundamentals of the testing methods and the main fields of application.

Due to the differences in the specific resistance to electric currents of rock, soil, voids, injected grout, solid concrete, etc., the distribution of the various materials can be determined horizontally and also vertically to considerable depth. This way it is not only possible to establish the amount and location of cracks or fissures in the rock of a dam abutment, or the location of voids in the subsoil under a concrete road pavement, or the location of cracks and defects in portions of an existing concrete structure, but also to determine the effectiveness of the subsequent cement or grout injection. All measurements are performed without physical injury to the structure; they can be coordinated with exploratory borings or any other control method, and since they are fast and economical they are very well adapted to provide detailed information over entire areas located, e.g., between check borings placed at greater distances.

Dynamics of the ball mill

Proceedings, Institution of Mechanical Engineers (London), V. 170, No. 23, 1956, pp. 773-800 (discussion, pp. 800-812)

Reviewed by Aron L. Mirsky

Part I—Power requirements based on the ball and shell system

H. E. Rose and D. E. Evans, pp. 773-783

Experiments on small-scale models and dimensional analysis were used in studying the power requirements for mills in operating condition (including balls) but without the powder charge. It was found that mills with lifters and mills without lifters differ so markedly they should be handled as two distinct cases. Results are presented in the form of charts or analytical expressions, and their use to calculate the power required to drive a mill actually grinding powder is demonstrated and shown to be reasonably accurate.

Part II—The influence of the powder charge on power requirements

H. E. Rose and D. E. Evans, pp. 784-792

Further experiments and investigations on mills actually grinding powder. Results are presented in a form which may be used to calculate, with engineering accuracy, the power required to operate a ball mill filled with granular material being dry-ground.

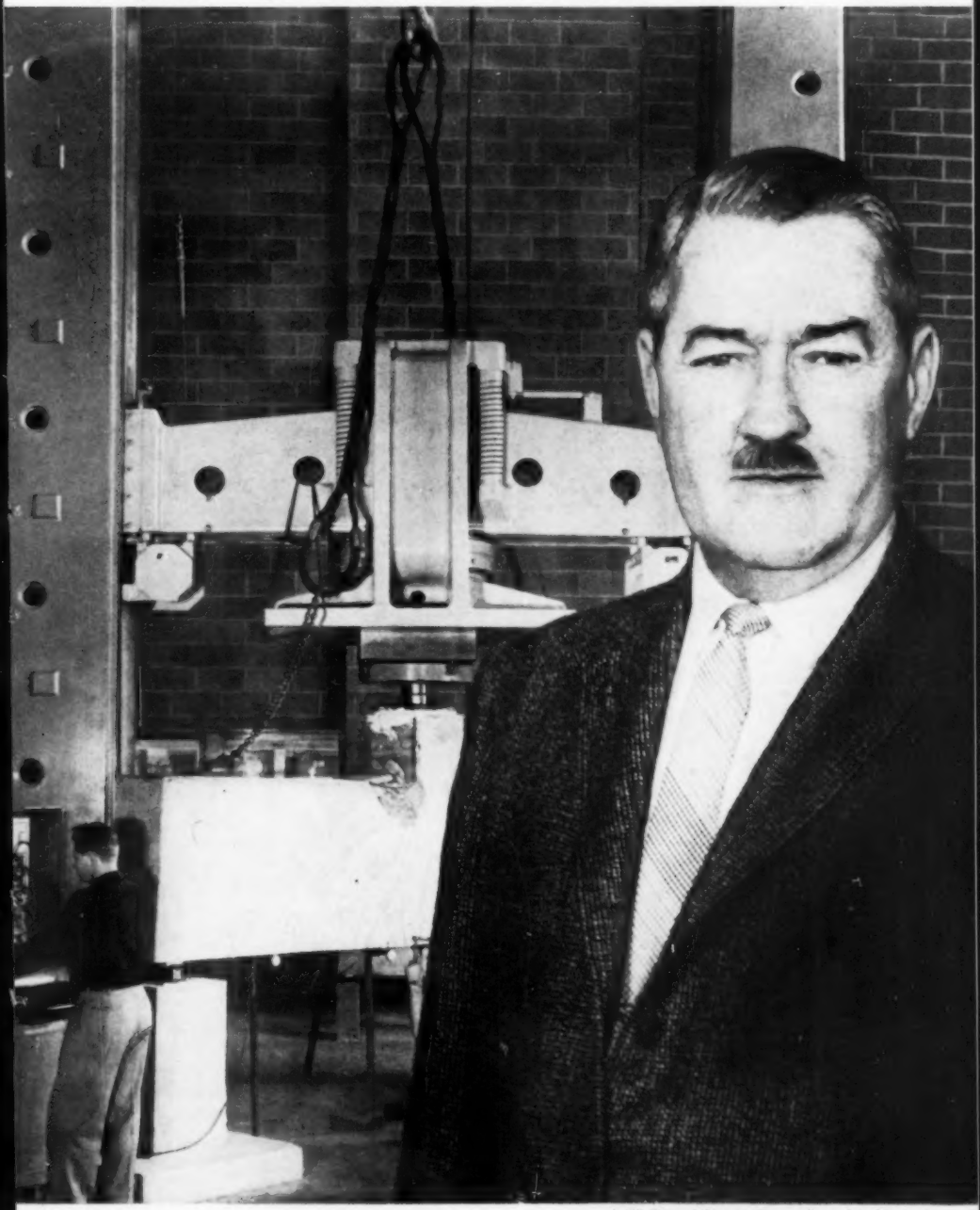
Part III—An investigation into the surging of ball mills

H. E. Rose and G. D. Blunt, pp. 793-800

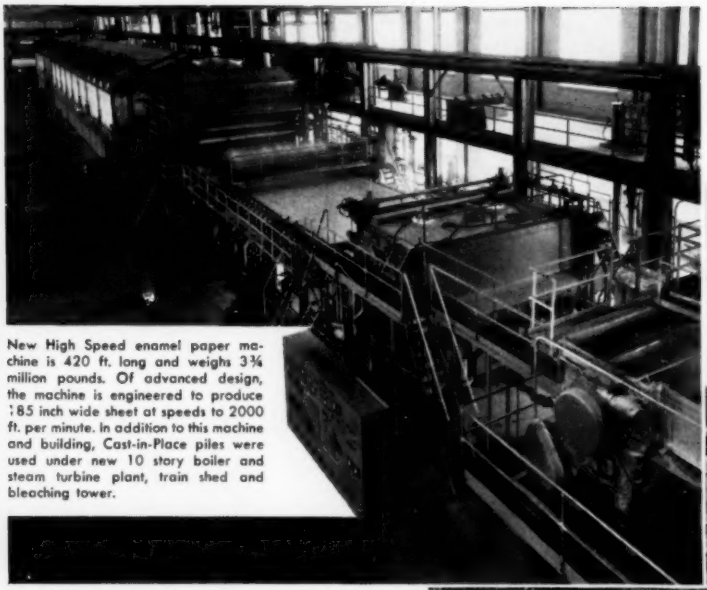
Presents results of experiments leading to a simple criterion to enable a designer to determine, in the design stage, whether a proposed mill will operate without surging. Authors suggest that, for economy, mills which can be operated under non-surging conditions be fitted with plain liners; for surging conditions, heavy lifters should be used to suppress the surging, even though the cost is increased.

ACI News Letter

Vol. 29, No. 10 JOURNAL of the AMERICAN CONCRETE INSTITUTE April 1958



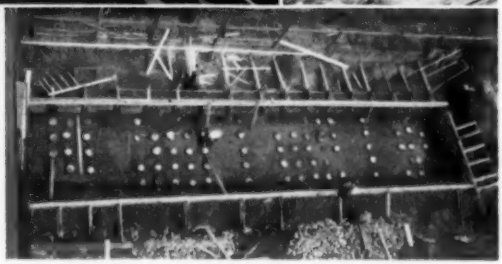
ACI President, Douglas McHenry



New High Speed enamel paper machine is 420 ft. long and weighs 3¾ million pounds. Of advanced design, the machine is engineered to produce 185 inch wide sheet at speeds to 2000 ft. per minute. In addition to this machine and building, Cast-in-Place piles were used under new 10 story boiler and steam turbine plant, train shed and bleaching tower.

Cast-in-Place rig places pile close to existing building without danger of vibration damage.

paper production boosted on Cast-in-Place piles



Cast-in-Place pile pattern for section of new plant.
(Photos—courtesy Consolidated Water Power & Paper Co.)

In planning new buildings for a 15 million dollar expansion program, Consolidated Water Power & Paper Company was confronted with difficult soil conditions at their Biron, Wisconsin, plant.

Borings indicated the existence of a non-uniform sand, clay, and decomposed granite soil underlain by granite bedrock at irregular depths. A further complication was the periodic rise and fall of the nearby river, causing a fluctuating water table and consequent movement or loss of fines in some areas. Under these conditions, it was considered inadvisable to depend on spread footings for buildings or high-load machine foundations. It was recommended that all loads be transferred directly to bedrock through the use of economical Cast-in-Place INTRUSION mortar piles.

INTRUSION-PREPAKT Cast-in-Place piles offered advantages not obtainable with any other type. Highly mobile equipment permitted operations close to existing buildings without danger of vibration or ground heaving usually associated with driven piling. In areas where soil was not self-supporting, casing was unnecessary because INTRUSION mortar under pressure fills hole as auger is withdrawn. This pressure also provided some lateral penetration to stabilize weak soil zones and increase pile skin friction. In all, some 1300 high-strength Cast-in-Place piles were placed for Consolidated's expansion program that added 230,000 square feet of floor space.

Cast-in-Place piles have a proven record of success all over the world. In addition to foundations, they provide a practical solution to many shoring, underpinning, stabilization and cofferdam problems. For more information on Cast-in-Place piles and other I-P construction services, contact: INTRUSION-PREPAKT, INC., 568-M Union Commerce Bldg., Cleveland 14, Ohio. In Canada, INTRUSION-PREPAKT, LTD., 159 Bay Street, Toronto, Ontario.

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ACI

News Letter Contents

April 1958

On the Cover—Douglas McHenry, director of development, Research and Development Division, Portland Cement Association, Chicago. Mr. McHenry assumed presidency of ACI at the recent convention in Chicago. Full story of meeting begins on p. 3.

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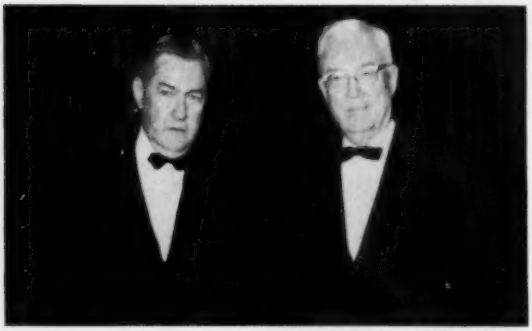
Concrete in Chicago . . .

ACI's 54th annual convention; contractors, producers, and civil engineers also met in the Windy City

Located close to the heart of the United States, geographically, Chicago was the undisputed center of activities for the civil engineering profession and concrete construction industries in February. Official registration for the ACI 54th annual convention, February 24-27 at the Morrison Hotel, totaled 784, with many visitors in attendance from the American Society of Civil Engineers and the National Concrete Contractors Association which were meeting concurrently in Chicago. Just ahead of the ACI convention, Chicago had played host to the National Ready Mixed Concrete Association, National Sand and Gravel Association, National Concrete Masonry Association, and the National Crushed Stone Association.

The ACI gathering devoted one full day to technical committee meetings. The three following days featured seven technical sessions on standards, the St. Lawrence seaway and power projects, fatigue of concrete, construction developments, design of highway pavements, structural design, and research. The structural design session was a joint program with the ASCE Structural Division.

Main attraction of the National Concrete Contractors Association meeting was the trade show at the Chicago Coliseum. Roger H. Corbetta, president of Corbetta Construction Co., New York, delivered the principal address at the opening session of this first annual convention of the group. Mr. Corbetta remarked that "concrete is coming into its own faster than we realize; and that the race is on to see how far and how high we can go in concrete." Because of this accelerating growth, Mr. Corbetta said it behooved the concrete contractor to belong to the American Concrete In-



Courtesy Engineering News-Record

Presiding officers . . . ACI's new president Douglas McHenry (left) joined with ASCE President Louis R. Howson of Alvord, Burdick, & Howson, Chicago, in welcoming guests at the joint ACI-ASCE banquet February 26 in Chicago

stitute. He deplored the fact that "a pitiful number belong," and urged NCCA members to sit in on ACI sessions at the Morrison Hotel.

Most of the contractors' association program was devoted to the formation of working committees and task forces necessary to begin operations for the recently formed national organization. The announced objectives of the new association include the establishment of uniform standards within the industry, the promotion of better business relations between contractors and suppliers, and to increase contractor members knowledge of products and methods.

Various joint social and technical activities were held by the Institute and ASCE. The joint structural session was the major activity. Several of the ASCE divisions also sponsored sessions and reports of particular interest to ACI members included a session on structural aspects of nuclear reactor design, a highway division session, and the engineering mechanics session. ASCE also offered a session devoted to electronic computation, and various reports from the air transport division, waterways and harbor division, and construction division were of special interest to ACI members.

Working Sessions for ACI Committees

More than 40 ACI technical committees and subcommittees scheduled meetings during the 54th annual convention, most of these being held on Technical Committee Day, Monday, February 24. The "working sessions" were open to the ACI membership in most cases and the committee meetings drew large attendance. The ACI Board of Direction, Technical Activities Committee, and the ACI Building Committee also met during the convention.

The committees and subcommittees that held meetings were as follows: Committee 115, Research, George W. Washa, chairman; 116, Nomenclature, Richard C. Mielenz, chairman; 201, Durability of Concrete in Service, Hubert Woods, chairman; 207, Properties of Mass Concrete, I. L. Tyler, chairman; 208, Bond Stress, C. P. Siess, chairman; 212, Admixtures, Bruce E. Foster, chairman.

Three subcommittees of Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete, met. They were: Subcommittee 1, Lightweight Structural Concrete, Ralph W. Kluge, chairman; Subcommittee 2, Lightweight Concrete Masonry, Clifton C. Carlson, chairman; Subcommittee 3, Insulating Concrete, Rudolph C. Valore, Jr., chairman.

Other committees meeting were Committee 215, Fatigue of Concrete, Gene M. Nordby, chairman; 314, Rigid Frames for Buildings and Bridges, James Michalos, chairman; 315, Detailing Reinforced Concrete Structures, James N. DeSerio, chairman.

Five subcommittees of Committee 318, Standard Building Code, under the chairmanship of Raymond C. Reese, met in addition to the meeting of the full committee.

The subcommittees were: Subcommittee 6, Design–General Conditions, Flexural Computations, headed by George E. Large; Subcommittee 8, Shear and Diagonal Tension, Bond and Anchorage, with C. A. Willson as chairman; Subcommittee 11, Columns, L. C. Urquhart, chairman; Subcommittee 12, Footings, Paul Rogers, chairman; and Subcommittee 15, Ultimate Strength Design, George Winter, chairman.

Committee 322, Design of Structural Plain Concrete, Paul Rogers, chairman, also met. Other committees were: 325, Structural Design of Concrete Pavements for Highways and Airports, E. A. Finney, chairman; 326, Shear and Diagonal Tension (joint ACI-ASCE), Charles S. Whitney, chairman; 331, Structures of Concrete Masonry Units, S. H. Westby, chairman. One subcommittee (of Committee 331) under the chairmanship of R. C. Sandberg, also met.

Other committees meeting were: Committee 332, Recommended Practice for Residential Concrete Work, Frank G. Erskine, chairman; 333, Design and Construction of Composite Structures (joint ACI-ASCE), Ivan M. Viest, chairman; 334, Concrete Shell Structures, Anton Tedesko, chairman; 401, Specifications for Structural Concrete, George H. Nelson, chairman; 605, Hot Weather Concreting, Stanton Walker, chairman; 609, Compaction of Concrete by Mechanical Means, Joseph J. Waddell, chairman; 611, Inspection of Concrete, L. H. Tuthill, chairman; 612, Recommended Practice for Curing Concrete, G. E. Burnett, chairman.

Two subcommittees of Committee 613, Recommended Practice for Proportioning Concrete Mixes, met. The subcommittee on proportioning lightweight aggregate concrete is under the chairmanship of J. J. Shideler; the subcommittee on no-slump concrete is headed by George W. Washa. Also meeting were Committee 614, Recommended Practice for Measuring, Mixing, and Placing Concrete, L. H. Tuthill, chairman; 616, Coatings for Concrete, John L. Rohwedder, chairman; 621, Aggregates, William R. Waugh, chairman.

Committee 622, Formwork for Concrete, under the chairmanship of Harry Ellsberg, met, as well as Subcommittee 1, Pressure on Formwork, led by W. R. Waugh; and Subcommittee 2, Construction Practices, J. R. Proctor, Jr., chairman. Other committees were 711, Precast Floor Systems for Houses, Leo M. Legatski, chairman; 714, Concrete Bins and Silos, C. H. Scholer, chairman; and 716, High-Pressure Steam Curing, S. B. Helms, chairman.

The Reinforced Concrete Research Council also met during the convention.

Four New Standards for ACI

Four new standards of recommended practice for the concrete industry were adopted by the American Concrete Institute at its 54th annual convention. Two of the standards affect pavement design and construction. One is on precast floor and roof units, and the other is a test procedure to determine relative bond value of reinforcing bars. As adopted by the convention, the standards now go to the general membership of the Institute for letter ballot to be canvassed June 1, 1958.

Concrete pavement standards

One standard is concerned with the design of concrete pavements, and the other with construction of concrete pavements and concrete bases.

"Recommended Practice for Design of Concrete Pavements," prepared by ACI Committee 325 under the chairmanship of E. A. Finney, director, Highway Research Laboratory, Michigan State Highway Department, Lansing, presents recommendations for the design of rigid concrete pavements and bases based on practice proved successful in the United States.

It offers comprehensive directions for design of rigid airport and highway pavements or bases for conditions of climate, traffic, available construction materials and equipment, and construction methods of the United States. It includes recommendations for soil foundations, selection of slab dimensions,

joints, and details, for reinforced or non-reinforced pavement. Recommendations for design of cement-stabilized bases, continuously reinforced pavement, prestressed pavement, and rigid resurfacings are not included since their use has not yet developed a generally applicable practice. Design practices of all principal state and federal agencies concerned with paving have been reviewed and related within the limits recommended.

(See Febraury 1957 JOURNAL, p. 717, for complete proposed standard.)

"Specifications for Concrete Pavements and Concrete Bases," prepared by ACI Committee 617 under the chairmanship of H. F. Clemmer, materials engineer, Engineering Department, Department of Highways, District of Columbia, Washington, D. C., will supersede the 1951 ACI standard on the same subject. (The proposed standard was published in April 1957 JOURNAL, p. 917.)

The specifications apply to construction of portland cement concrete pavements and bases under normal conditions for both highways and airports. Standards and specifications of several other organizations are incorporated by reference.

Sources and acceptance standards for materials are given, as well as materials testing procedures and procedures for test of concrete strength to be used as a basis for proportioning of mixtures. Specifications for the materials and construction of soil foundations for concrete pavements and concrete bases are included. Provision is made for use of foundations stabilized by a cementing agent, but materials and construction of such are beyond scope of this specification.

Materials, dimensions, setting, and removal of forms are treated. Construction methods are specified for forming joints, installation of joint seal and load transfer devices, and placing of reinforcement. Concrete proportioning based on design for minimum strength is covered in detail; proportioning based on fixed cement content is allowed.

Other sections cover production of high-early-strength concrete and the handling and mixing of materials. Detailed requirements are given for placing, fininshing, and curing of pavement concrete. Check of thickness of finished pavement is cited as basis for adjustments in payment to contractor. Protection of finished pavement, opening it to traffic, and public use of thoroughfare during construction are also specified.

Precast concrete floor and roof units

Various types of precast concrete floor and roof units have been developed and used for office and school buildings, apartments, residential structures, and similarly loaded structures. These precast reinforced concrete floor systems include the I-beam and hollow core joist, precast block assembled to form a "plank" or "slab," precast joist with a precast filler placed between, precast slabs supported by two integrally precast T-joists, and various other combinations. Most of such units are produced under closely controlled factory conditions and then shipped to the job site for quick assembly and erection in the over-all structure.

"Minimum Standard Requirements for Precast Concrete Floor and Roof Units," prepared by ACI Committee 711 under the chairmanship of Leo M. Legatski, professor of civil engineering, University of Michigan, Ann Arbor, supersedes the 1953 ACI standard on the same subject. (The proposed standard was published in December 1957 JOURNAL, p. 441.)

The minimum standard requirements for single units or multiple element assemblies are meant to be used in conjunction with the ACI Building Code (ACI 318-56) which has been incorporated in many municipal building codes. The new standard on precast units covers materials; design principles; manufacture, including curing and handling; testing of completed units; installation plans; and special provisions for holes and openings in members. The design chapter treats such problems as dimensions, allowable deflection, structural concrete topping, reinforcement anchorage and location, and use of lightweight concrete.

Determining bond value of reinforcing bars

"Test Procedure to Determine Relative Bond Value of Reinforcing Bars," prepared by ACI Committee 208 under the chairmanship of C. P. Siess, research professor of civil engineering, University of Illinois, Urbana, provides a uniform basis for comparison of

ACI humorist H. J. Gilkey was on the other end of a good story this time. He is shown here with Arsham Amirikian (right), 1958 recipient of the Alfred E. Lindau Award. Professor Gilkey received the Institute's 1958 Turner Medal

bond qualities of different reinforcing bars. Prof. H. J. Gilkey, theoretical and applied mechanics department, Iowa State College, Ames, long-time chairman of Committee 208, presented the proposed standard to the convention.

The recommended test method uses companion beam type specimens, cast horizontally in inverted and normal positions so that the effects of water gain and settlement of concrete are included in the evaluation. The concrete surrounding the bars is in tension as bars are ordinarily used, which makes the results more acceptable to some users. Minimum criteria for acceptance are not included since the purpose is merely to establish relative bond values for the different bars under consideration.

The test may be used to evaluate the effectiveness of deformed bars having characteristics other than those described in "Tentative Specifications for Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement," ASTM A 305-56T. For construction under the ACI Building Code, an evaluation of such bars is necessary since the code definition for a deformed bar is one which meets the requirements of ASTM A 305. Manufacturers of bars meeting the requirements of ASTM A 305 may use the test procedure for product improvement and to evaluate modifications in the design of deformations.

Committee 208 amended the proposed standard at the time of presentation, the amendment being accepted by the convention. The amendment was thought desirable since it presents background information of value to the user. As amended, the following "introduction" is to appear at the beginning of the test procedure immediately following the synopsis (see ACI JOURNAL, August 1957, p. 89).

INTRODUCTION

Previously published identically as information,[1] this test procedure had a thorough workout through its use in extensive testing programs culminating in the evolution of specifications covering minimum geometric requirements.[2] Acceptance by the ACI Building Code committee[3] of the ASTM A 305 definition of a deformed bar produced an immediate drastic change in both structural and general practice, resulting in the almost immediate disappearance of old-style nonconforming deformed reinforcement.

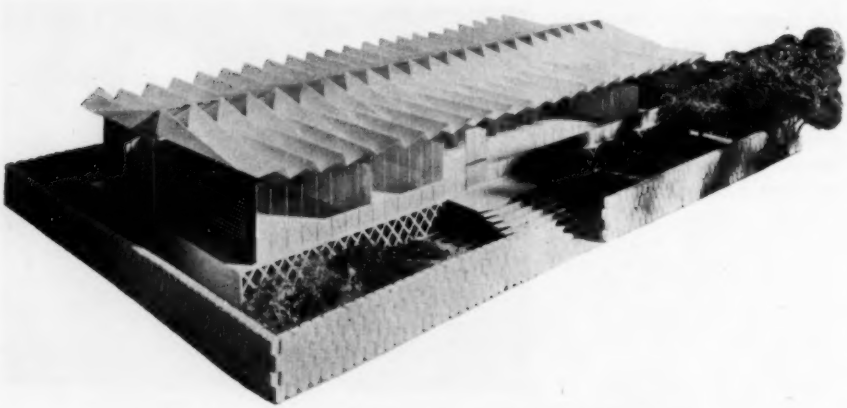
Background considerations involving alternative possibilities, controversial aspects, and special credits due to individuals and organizations are covered in the foreword and discussions that accompany the 1945 publication of the test procedures.[4]

[1]ACI JOURNAL, Feb. 1945, *Proc.* V 41, pp. 273-292, with discussion in ACI JOURNAL, Nov. Supplement 1945, *Proc.* V. 41, pp. 292-1–292-8.

[2]ASTM Designation A 305.

[3]"Building Code Requirements for Reinforced Concrete (ACI 318-51)," ACI JOURNAL, Apr. 1951, *Proc.* V. 47, pp. 589-652.

[4]See Reference 1, pp. 273-277 and 292-1–292-8.



New ACI Building Spotlighted

The opening convention session, February 25, with President Walter H. Price, presiding, also featured a report and movie on the new ACI headquarters building. Minoru Yamasaki, Yamasaki, Leinweber & Associates, Birmingham, Mich., architect for the building, stated: "We believe that concrete is a very wonderful and versatile material for buildings if it is used in a technological way. We have tried to demonstrate this versatility in our work on the ACI building." He showed pictures and described the new building, and also illustrated his use of concrete in various other structures, particularly in shells and folded plate construction. Structures cited included an airport terminal building, a cultural center for a university, and several unique warehouses.

The session closed with a colored movie, narrated by Paul Rice, ACI technical director, showing the construction and erection of the precast folded plate concrete roof on the ACI headquarters building.

Local Chapters Authorized by the Revised ACI Bylaws

As adopted at the 54th annual convention, the major change in the American Concrete Institute Bylaws is the addition of a section empowering establishment of local chapters of the Institute. The amended Bylaws now go to the ACI membership for letter ballot.

To provide a means of advancing the interests of the Institute in a specified geographical area and of furthering the chartered objectives for which the Institute is organized, local chapters may be authorized by the Board of Direction under the new Bylaws. Organization of a chapter may be authorized on the written request of 25 members of the Institute residing within the proposed chapter area, provided that at least 50 members reside in the area it is proposed to encompass. Each Institute member shall be deemed

eligible to belong to a chapter if his permanent address of record at Institute headquarters is within the confines of that chapter area.

A provisional chapter has been in operation for almost a year—the Southern California Chapter in Los Angeles. ACI members in several other areas in the United States now contemplate petitioning the Board of Direction for establishment of other local chapters.

The other changes adopted at the convention (see "News Letter" section, ACI JOURNAL, Dec. 1957, pp. 32-34) were primarily for clarification of the Bylaws.

St. Lawrence Seaway and Power Projects combine construction techniques of two countries

The St. Lawrence International Seaway and Power Projects are an outstanding example of political, engineering, and construction cooperation between two countries. Construction-wise, it is a unique effort in that in one undertaking the United States and Canada are each using their own methods of construction. One complete session, sponsored by ACI Committee 207, Properties of Mass Concrete, was devoted to this joint effort, with emphasis being placed on the concrete work involved in the power project and in the seaway project. I. L. Tyler, manager, Field Research Section, Portland Cement Association, Chicago, and chairman of Committee 207, presided.

The possibility of constructing the St. Lawrence River deep waterway has for many decades captured the imagination of Canadians and Americans. If the river could be deepened in certain places, notably between Cornwall and Montreal, and if the limitations of the 14-ft Cornwall Canal, which by-passed the International Rapids could be overcome, then larger ocean-going ships would be able to penetrate the Great Lakes to the very heart of the continent, to the economic advantage of Canada and the United States. The International Rapids also pointed up another possibility of the river—water power. There is about a 90 ft drop in the water level between Lake Ontario and Cornwall.

J. N. Mustard, assistant engineer-in-charge, Concrete Control Department, Research Division, Hydro-Electric Power Commission of Ontario, Toronto, described features of the St. Lawrence International Power Project. Concrete technology on the power project was described by Harry H. McLean, director, Public Works Laboratory, New York Department of Public Works, Albany. Features of the St. Lawrence International Seaway were covered by Gordon Kidd, concrete control engineer, St. Lawrence Seaway Authority, Montreal, and concrete technology on the seaway project was described by M. R. Smith, Office of the Chief of Engineers, Department of the Army, Washington, D. C.

Power project

In the International Rapids section of the river, a $600,000,000 power project is being built by Hydro-Electric Power Commission of Ontario and the Power Authority of the State of New York. When construction is completed in 1960, the state of New York and the province of Ontario will share equally 1,640,000 kw of electricity. The three major structures are the powerhouse and dam on Barnhart Island, Long Sault Dam, and Iroquois Dam. Numerous minor

structures are involved, as well as relocation and rebuilding of several villages.

In pointing out the differences in materials, methods, and concrete mixes used on the power dam, on which both American and Canadian contractors are working, it was noted that the basic concrete mix proportioning requirements for the Canadian side were primarily the same as the American mixes, the main differences being in the exterior concrete. The Americans used 6 in. maximum size aggregate as compared to 3 in. maximum size aggregate that was used by the Canadians.

The Americans used a blend of portland and natural cement in Barnhart Power Dam interior concrete, while the Canadians used an all portland mix, except where heat reduction for shrinkage control was considered imperative; there they used fly ash as a partial cement replacement. The Americans specified and used Type II cement, while the Canadians used the "Canadian Standard" cement comparable to the U. S. Type I.

In transporting and placing concrete, all transportation of concrete on the American side was by bucket or dumpster; conveyor belts were not permitted. On the Canadian side, conveyor belts were used for transporting all concrete from the plant.

In general, all mass concrete on the American side was limited to a 5-ft lift, while on the Canadian side lifts were permitted up to 35 ft. The Americans used steel forms, the Canadian used wood. The Americans used horizontal joints with no waterstops; the Canadians used 8-in. keyways and horizontal waterstops.

The final accomplishments were the same—schedules were met on time, and the international structure is taking its final form, ready to serve both nations in equal capacities.

Medalists

Phil M. Ferguson, left, received the Wason Medal for Research, and R. E. Copeland was awarded the Wason Medal for the Most Meritorious Paper. Mr. Ferguson, who continues in office as Institute vice-president, reported his work in "Some Implications of Recent Diagonal Tension Tests." Mr. Copeland authored "Shrinkage and Temperature Stresses in Masonry."

Seaway project

The seaway construction encompasses numerous locks, dredging and excavating canals, relocation of highways and railroads, modification of bridges, construction of dikes and retaining walls—with some of the work being coordinated with various power projects on the Canadian and American sides of the border.

In general, concrete was mixed and placed by standard methods. Differences in methods used were those caused or permitted by differences in the specifications.

In placing the concrete, specifications of the Canadian St. Lawrence Seaway Authority restricted dropping the concrete to 4 ft. The Corps of Engineers specifications (construction agency for the St. Lawrence Seaway Development Corporation, which is the United States agency) restricted dropping the concrete to 5 ft. The thickness of layers specified by the Canadians was 12 in. and by the Corps of Engineers, 20 in. A major difference was in the height of lift. The Corps of Engineers restricted the height of lift to 5 ft, except where the cross section of the monolith was 16 ft or less in width, in which case the height of lift was restricted to 10 ft. The Canadian Seaway Authority did not restrict the height of lift. The Corps required that 120 hr must elapse between lifts;

no time limitation was imposed by the Canadians.

Another major difference existed in curing. The Corps required that concrete be cured by keeping it moist for 14 days. Curing compounds were permitted only in isolated cases. The Canadians required moist curing for 7 days. Curing compounds approved by the engineer could be used.

The 120-hr delay between placing successive lifts and the 5-ft lifts forced the U. S. contractors to spread out their operations more than the Canadians. The 14-day curing period and the 5-ft lifts required by the Corps also increased the costs of winter protection on their projects. Practically all the concrete on the Corps projects was placed during periods when heating would not be required. Their production was geared to complete concrete construction during warm weather. The Canadians, however, placed concrete through the winter months.

Fatigue of Concrete

Five reports concerned with fatigue of concrete were presented at the session sponsored by ACI Committee 215. Gene M. Nordby, program director for engineering sciences, National Science Foundation, Washington, D. C., and chairman of Committee 215, presided at the session. The papers are summarized below and it is planned that a future issue of the ACI JOURNAL will be devoted to this symposium.

Fatigue of Concrete: A Review of Past Research—GENE M. NORDBY, program director for engineering sciences, National Science Foundation, Washington, D. C.

The history of fatigue of concrete investigations is reviewed starting in 1898 with the work of Considére and De Joly. Over 100 publications on the subject, including those in the foreign literature, were surveyed. The most important investigations are summarized and the salient facts which seem to be emerging from the research are listed. The work has been divided for discussion into six categories: fatigue in compression, fatigue in flexure, fatigue in tension, fatigue of bond, fatigue of reinforced concrete, and fatigue of prestressed concrete.

Most of the research up to this time has been exploratory. More research is needed in every phase of fatigue of concrete and future investigations should be well organized to isolate a particular variable. Research on the fundamental properties of concrete fatigue to describe the mechanism of fatigue failures may be particularly fruitful. The potential economic return from evaluation of fatigue problems is almost fantastic. The saving in highway construction alone would be enormous if the life of concrete pavements could be prolonged 10 years.

Effect of Range of Stress on Fatigue Strength of Plain Concrete Beams—JOHN W. MURDOCK and CLYDE E. KESLER, respectively, instructor and professor, Department of Theoretical and Applied Mechanics, University of Illinois, Urbana

From the results of tests of 175 plain concrete beams subjected to repeated flexural loading, it was found that plain concrete exhibits no fatigue limit when subjected to loads which produce no reversals of stress. Fatigue strengths at ten million repetitions of stress were determined for each of the several ranges of stress investigated, and

Presidential Address

Full text of address delivered by retiring President Price at the Awards Luncheon, February 26, reviewing 1957 accomplishments and activities of the Institute and highlighting events in the past ten years of ACI's development, appears on p. 825 of this month's JOURNAL.

DELAWARE HIGHWAY DEPARTMENT SELECTS PRESTRESSED CONCRETE FOR NEW BRIDGE...

Above: Placing the last of the first five spans of the new Fenwick Island Bridge. There will be eleven spans in all with a total length of 440′. Box beams are 39′ 11″ x 3′ wide and weigh about 10 tons. Each span requires 13 beams.

Owner: Delaware State Highway Department

Designed by: Bridge Division, Delaware State Highway Department

Contractor: George & Lynch, Inc., Wilmington, Del.

Prestressed Beams Manufactured by: Eastern Prestressed Concrete Corporation, Line Lexington, Pa.

At right: Moving beam from casting bed in background to stockpile, early in the morning at manufacturer's plant. Beam was cast the previous day.

This new bridge links Fenwick Island with the mainland in the famous Delaware-Maryland Seacoast area. Its spans are prestressed concrete, chosen by the Delaware Highway Department for quicker, more convenient construction, at lower costs.

Precast to exacting specifications at the manufacturer's plant, the box beams were trucked to the job site as needed. Beams were quickly lifted from the truck and swung into place by crane—making it easy to place a span a day.

In manufacturing these beams, Eastern Prestressed Concrete Corporation used Lehigh Early Strength Cement to gain maximum production efficiency. With the concrete reaching 4000 PSI in 18 hours, it was possible to make 8 beams every other day using only one 345′ casting bed.

This is typical of the advantages of Lehigh Early Strength Cement in modern concrete construction.

LEHIGH PORTLAND CEMENT COMPANY

Allentown, Pa.

these strengths were found to be dependent on the range of stress to which the specimens were subjected.

Test results agree well with previous investigations insofar as valid comparisons may be made. The need for additional research is, however, most apparent; results from this investigation do not permit definition of the fatigue behavior of plain concrete subjected to reversals of stress, nor do they permit a valid description of the process by which fatigue failures occur.

Probability of Fatigue Failure of Plain Concrete—JOHN T. MCCALL, associate professor, Applied Mechanics Department, Michigan State University, East Lansing

Fatigue tests were run on concrete beams and the data were analyzed in an attempt to determine the relationship for concrete between stress S, number of cycles to failure N, and probability of failure P.

Prismatic concrete beams were loaded in completely reversed bending at the rate of 1800 cycles per min. Beams were tested at two stress levels and the number of cycles necessary to cause failure was determined. Testing limit was twenty million cycles.

From the data for tests run to ten million cycles, it was concluded that the S-N-P relationship for concrete can be obtained graphically and that one of the mathematical relationships is valid for the concrete tested. It was also concluded that a definite fatigue limit for the concrete studied does not exist (in a range up to ten million cycles) and it was estimated that the probability of concrete exceeding a fatigue life of ten million cycles is one-half—if the alternating stress amplitude is one-half of the ultimate strength and the mean stress is zero.

Continued on p. 14

Chicago Registration Records

Illinois provided the largest group of registrants attending the ACI 54th annual convention held in Chicago in February,

In addition to 40 other states and Canada being represented there was also Taipei, Taiwan, represented by James T. M. Chwang, Continental Engineering Corp.; Havana, Cuba, represented by Jose A. Vila, Universidad de la Habana; Mexico City, Mexico, represented by Enrique Torres, Corrugated Bar Manufacturing Co.; Panama, Republic of Panama, represented by Celso A. Carbonell, Sika Panama S. A.; Johannesburg, South Africa, represented by Keeve Steyn, Consulting Engineer; and Bachaquera, Venezuela, represented by John G. Dempsey, Compania Anonima de Concreto.

Following are registration totals from the several states and countries.

Illinois	254
New York	60
Canada	56
Michigan	40
Pennsylvania	38
Ohio	37
Wisconsin	35
California	32
Washington D. C.	27
Indiana	25
Missouri	23
Texas	19
Colorado	16
Kentucky	10
New Jersey	10
Georgia	9
Iowa	9
Tennessee	8
Massachusetts	7
Minnesota	7
Virginia	7
Florida	6
Oklahoma	6
Kansas	5
Mississippi	4
Connecticut	3
Delaware	3
Louisiana	3
Nebraska	3
North Carolina	3
Washington	3
West Virginia	3
Alabama	2
Idaho	2
Maine	2
Maryland	2
South Carolina	2
South Dakota	2
China	1
Cuba	1
Mexico	1
New Mexico	1
North Dakota	1
Panama	1
South Africa	1
Utah	1
Venezuela	1
Wyoming	1

Continued from p. 13

Static and Fatigue Strength of Beams with Tensile Reinforcement—TIEN S. CHANG, Department of Applied Mechanics, Virginia Polytechnic Institute, Blacksburg, and CLYDE R. KESLER, professor, Department of Theoretical and Applied Mechanics, University of Illinois, Urbana

This paper studies both the static and fatigue strength in shear and in flexure of concrete beams with tensile reinforcement, and the transition characteristics of the modes of failures in shear and in flexure under the repeated loading.

Formulas were developed to estimate the static initial diagonal cracking load and the static ultimate moment of beams with tensile reinforcement. Fatigue tests were conducted on simply supported concrete beams of relatively small cross sections (4 x 6 in.) with tensile reinforcement only. Statistical studies of the fatigue behavior with regard to initial diagonal cracking, shear-compression, and fatigue of reinforcement were made.

The interactions of initial diagonal cracking, and failure due to shear compression or fatigue of the reinforcement were found to be extremely complicated.

Fatigue Properties of Concrete Beams—THOMAS E. STELSON, associate professor and acting head, Department of Civil Engineering, Carnegie Institute of Technology, Pittsburgh, and JOHN N. CERNICA, assistant professor, Department of Civil Engineering, Youngstown University, Youngstown, Ohio

Eleven concrete beams with regular tensile reinforcement were tested under repeated load applied 320 times per min. The beams were identical except for a slight increase in concrete strength during the period of testing. The ACI elastic-design load for the test beams was 2730 lb. The ultimate load was 8800 lb. The load causing fatigue failure after 500,000 cycles was 5720 lb. Thus, the ultimate load was 3.22 times the design load and the fatigue load was 2.09 times the design load. The test results show greater relative fatigue strength than has been previously reported and show that safety against fatigue failure is greater in beams than in compression members.

Construction Session

Running concurrently with the fatigue session on Wednesday morning was the session devoted to construction technology presided over by Douglas McHenry, director of development, Research and Development Division, Portland Cement Association, Chicago. The session included descriptions of two recent outstanding projects, a discussion of control of concrete mixes on a large highway project, and a historical survey of failures with lessons to be learned from them. The reports are summarized below.

Failures of Concrete Structures—JACOB FELD, consulting engineer, New York

A historical survey of concrete failures of the last half century in the United States; discusses significant examples without attempting to list all failures. Touches on legal aspects from the Code of Hammurabi through English common law with some observations on present code requirements as they relate to different types of failure. Concrete failures are grouped according to their major causes: design deficiency, drafting and detailing errors, concrete mix, supervision omission, frost protection defects, bearing wall deficiency, foundation deficiency, faulty erection techniques, temperature and shrinkage, secondary stresses, and inadequate formwork. (See also December 1957 JOURNAL, p. 449.)

Control of Concrete Mixes—EDWARD A. ABDUN-NUR, consulting engineer, Denver, and JOSEPH J. WADDELL, project materials engineer, Joseph K. Knoerle & Associates, Chicago

This progress report on concrete control on the Northern Illinois Toll Highway outlines

Triumvirate of Institute actives —ACI President Douglas McHenry, left, with A. Allan Bates (center), head of the local committee in charge of Chicago convention arrangements, who also heads the ACI Building Committee. On the right is retiring Institute president Walter H. Price

the basic features of the control program, and gives examples of the control attained. This project entails some 50 contracts under the supervision of 23 section engineers and a general consultant, all reporting to the Illinois State Toll Highway Commission.

Four classes of concrete are briefly described, and uniformity producing features of the specifications are discussed. In the investigation and approval of aggregate sources, petrographic examinations and service records are important features of the program. Procedures used in pre-testing cement, and in developing mix proportions are outlined. Operations of the field control laboratories and the central concrete laboratory are described, with particular emphasis on procedures used to increase uniformity.

Sampling of concrete batches, and making of cylinders for compressive strength tests are handled in such a manner as to permit a continuous check on the operations of each contract, and to permit the engineers to often forestall unsatisfactory concrete. Semi-automatic batch and central mix plants are used. These have automatic recording moisture meters, finish screening of coarse aggregates just prior to passing into the plant storage bins, and recorders to indicate the batch weights.

Several examples of control charts and a discussion of the significance of the variations in each case are presented. It is concluded that this type of control is practical, effective, expedites the work, motivates the contractor to provide a higher degree of uniformity, and in general improves the quality obtained without being burdensome.

Design and Construction of a 22-Story Office Tower—OTTO SAFIR, consulting engineer, Vancouver, B. C.

The 22-story office tower for the British Columbia Electric Co., Ltd., in Vancouver comprises two basements, ground floor, mezzanine, 21 upper floors, and penthouses. Because of its narrow width and the required resistance against wind forces and earthquakes, the main structural element is formed by the reinforced concrete core which houses the essential services, i.e., elevators, mail conveyors, service ducts, staircases and lavatories. This cellular core resists all horizontal forces and carries the greater part of the vertical loads. The structure of

3

IMPORTANT ADVANTAGES OF CONCRETE PAVEMENT CONTINUOUSLY REINFORCED WITH RAIL STEEL BARS

Low initial cost

EXAMPLE: On a recent highway project in Pennsylvania, the low bidder submitted these unit prices:

Continuous reinforced conc. pavement	$4.75 psy
Standard reinforced conc. pavement	$5.25 psy

Minimum maintenance

EXAMPLE: During ten years of normal use, a stretch of continuously rail-steel-reinforced pavement on US 40 in Illinois has shown no visual crack widening or corrosion damage despite 70° temperature drops.

More riding comfort

TRY IT YOURSELF. You'll find continuously reinforced pavements in New Jersey, Indiana, California, Texas, Illinois, Pennsylvania; planned for construction in Maryland, Virginia and Michigan.

For more details on continuously reinforced concrete pavement plus an interesting case history of its construction, we invite you to borrow, without charge, our 16mm color film, "Advancement in Concrete Pavement." Write or 'phone.

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the office sections surrounding this core is composite of structural steel and reinforced concrete to reduce the sections of the columns to a minimum and to achieve rapid construction. The method of analysis of the core is described along with construction procedure which resulted in the construction of one complete floor in seven working days.

Wide-Span Hyperbolic Paraboloidical Shell—ANTON TEDESKO, manager, Roberts and Schaefer Co., New York

Part of the new Courthouse Square Development in Denver is a shell 132x113 ft—believed to be the widest span hyperbolic paraboloidical shell in North America. The shell structure is made up of four hyperbolic paraboloidical surfaces and supported at four points located in the corners of a rectangle. The rise of the shell is 28 ft. The four hyperbolic paraboloidical surfaces are joined along ridge bands which form a big horizontal cross-Straight line marginal ribs provide the vertical gable framing along each of the sides of the rectangle. The shell has a minimum thickness of 3 in., gradually increases to 5 in.

Design features of the shell were illustrated, including appearance, loads, typical stresses, arrangement of ribs, rib bands, reinforcing steel hinges and buttresses. Construction features covered were the centering design, construction sequence, concreting plant, concreting operations, finishing, decentering procedure, and deflections. The concrete placing was complicated by 100 F temperatures and low humidity (20 percent).

Preview of Tomorrow's Highways

Will the pavements we design and build today be adequate 20 years hence? One complete session was devoted to previewing highways of the future based on present design requirements, construction practices, and experimental work now going on with concrete pavements. The session was sponsored by ACI Committee 325, Structural Design of Concrete Pavements for Highways and Airports, whose chairman is E. A. Finney, director, Highway Research Laboratory, Michigan State Highway Department, East Lansing. Mr. Finney introduced the session by reviewing briefly the work of Committee 325 and discussed design requirements now being used and/or studied in reference to concrete pavement.

The reports of three subcommittees of Committee 325 were featured. Subcommittee 6, headed by J. A. Bishop, U. S. Naval Civil Engineering Research and Evaluation Laboratories, Port Hueneme, Calif., reported on prestressed concrete pavements; Subcommittee 7 under the chairmanship of J. D. Lindsay, Illinois Division of Highways, Springfield, reviewed the use of continuous reinforcement in highway pavements; Subcommittee 8, whose chairman is L. M. Arms, Portland Cement Association, Chicago, reported on the design of concrete overlays for concrete pavements.

In view of the expanded highway program now in progress, it was emphasized that highway engineers should be thinking of present design requirements for concrete pavement in terms of 1980, the latest target date for completion of the interstate system. Experimental work now in progress will probably form the basis for pavements of tomorrow.

An increase in the number of continuously reinforced concrete pavements constructed is anticipated. Prestressed concrete has also proved feasible for pavement slabs in experimental projects. It is quite possible that some modification or combination of these types of construction will be the design of the future.

Permissible axle loads for commercial vehicles now range from 18,000 to 22,400 lb. The tendency to increase axle loads in certain areas will continue. Permits for operation of heavy hauling equipment is increasing rapidly

each year; axle loads as high as 35,000 lb on multiple wheel axles are becoming common. The frequency of axle load applications, a major factor in pavement design, is increasing in proportion to increase in traffic and travel trends. Between 1950 and 1956 there was approximately a 7 percent per year increase in ton-miles carried by commercial vehicles. This continuing trend could represent an imposing load to be carried by the pavements of 1980. All of these load factors must be considered in selecting design wheel loads—building today's pavements for tomorrow's use.

Trends in modern transport suggest that no road will be constructed less than 24 ft wide regardless of traffic conditions or cost. reduce maintenance cost and provide greater safety and mobility convenience to the highway user.

Prestressed concrete

A few experimental prestressed concrete pavements have been built in the United States, but most of the investigative work in the form of field slabs and tests have been undertaken in England and France. When the tensile strength of concrete is increased by inducing an initial compressive stress, a pavement slab so constructed will have a load capacity many times that of a conventional concrete slab of the same thickness on the same subgrade. There are two general methods by which the concrete may be "prestressed"; namely, through the use of post-tensioned cables or through the use of hydraulic jacks reacting against abutments. Prestressed pavement is now becoming economically competitive with conventional pavement. The cost differential will be reduced as contractors gain experience and techniques are perfected.

LOS ANGELES IN '59

ACI's Technical Activities Committee is already looking forward to the 1959 convention in Los Angeles; program plans are taking shape.

Those who intend to offer papers for presentation at the 55th annual meeting should write to Institute headquarters before July 1, furnishing a synopsis which should make clear the scope of the proposed paper and indicate features that the author thinks will justify its inclusion in the program. Contributors should have preliminary drafts of manuscripts in the hands of the Technical Activities Committee for appraisal and acceptance by Sept. 15, 1958, and final manuscripts of accepted papers will be due Jan. 1, 1959.

From the replies received and suggestions from other sources, TAC will select papers to make up each convention session.

Write Institute headquarters NOW about the papers YOU want to see on the 1959 program.

The transition from 12-ft lanes on expressways to 11-ft lanes on other systems creates a driving hazard; besides, the extra foot per lane adds materially to the structural strength of the pavement and thus to its economic life.

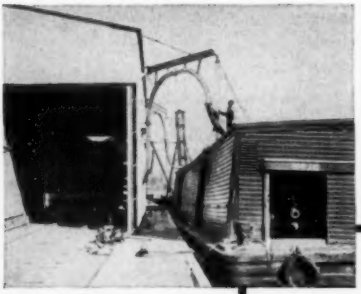
Another factor facing highway engineers is the rapidity with which concrete and other types of pavement surfaces become slippery when wet under modern traffic. The ultimate answer may be an entirely new concept in concrete pavement mixes, particularly for the top part of the slab.

Road tests indicate that paved shoulders have a definite beneficial effect on pavement performance. Paved shoulders should ultimately pay for themselves by extending the structural life of the slab and at the same time

Continuously reinforced concrete pavements

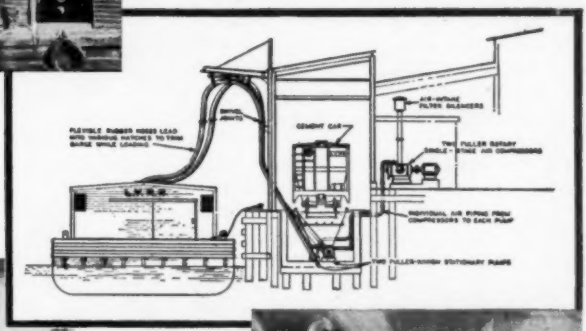
Subcommittee 7 described briefly the continuously reinforced pavements which are now under observation, discussed their performance, and summarized the present state of knowledge on the subject.

Over the past 40 years, pavement design has passed through a variety of phases: plain concrete pavement without joints, plain



Hoses being placed in hatches of barge. Cement car in shed over pumps in pit.

Fuller-Kinyon System unloads cars . . . conveys to barges . . . unloads barges for Lehigh Valley Railroad



Unloader behind bulkhead in end of barge.

Fuller-Kinyon Pumps in pit underneath car.

Bulk-cement handling by the Lehigh Valley Railroad at its Jersey City terminal is speeded up by the Fuller-Kinyon System. Fuller-Kinyon Pumps convey from cars to barge and Fuller-Kinyon Remote-Control Unloaders convey from barge to storage at various points in New York Harbor . . . speedy, efficient, time-saving handling from time cars arrive until final delivery to consumer.

Two stationary pumps in pit underneath car, convey through two lines to barge at rate of 900 to 1000 bbl. an hour. Unloading barge is accomplished at rate of 150 bbl. an hour. One man operates and controls all movements of the Unloader by a flexible-electrical cable and hand-control switch, wherever cement is to be picked up in the barge.

Air for the two stationary pumps is supplied by two Fuller Rotary Compressors, installed on the pier adjacent to the car-unloading station, at 614 c.f.m., 12-lb. pressure. Air for the Unloader is furnished by consumer's plant at destination.

Write for Bulletin FK-27, illustrating and describing this very interesting plant operation.

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concrete pavement with expansion joints, plain concrete pavement with both expansion and contraction joints, plain concrete pavement with contraction joints only, and reinforced concrete pavements with similar variations in the use of joints. Spacing of joints has varied from 15 ft to as much as 1000 ft.

Joints of various designs have been used with varying degrees of success but they continue to be points of potential structural weakness, present maintenance problems, and permit infiltration of water and soil. Often joints adversely affect the riding quality of pavement.

Many thousand of miles of pavement using the various joint types and spacing have served satisfactorily, but joint problems which were not readily solved have caused engineers to think about building continuously reinforced pavements, thereby eliminating closely spaced joints. It is quite likely that continuously reinforced concrete pavements will be standard design for future concrete highways.

Concrete overlays

Concrete overlays or concrete resurfacing have an experience record of widespread use in the United States. During 1957, a survey of all known street and highway concrete resurfacing projects included over 300 projects in 28 states; of these, 40 percent were rated as in excellent condition, 40 percent as good, 18 as fair, and only 2 percent as poor.

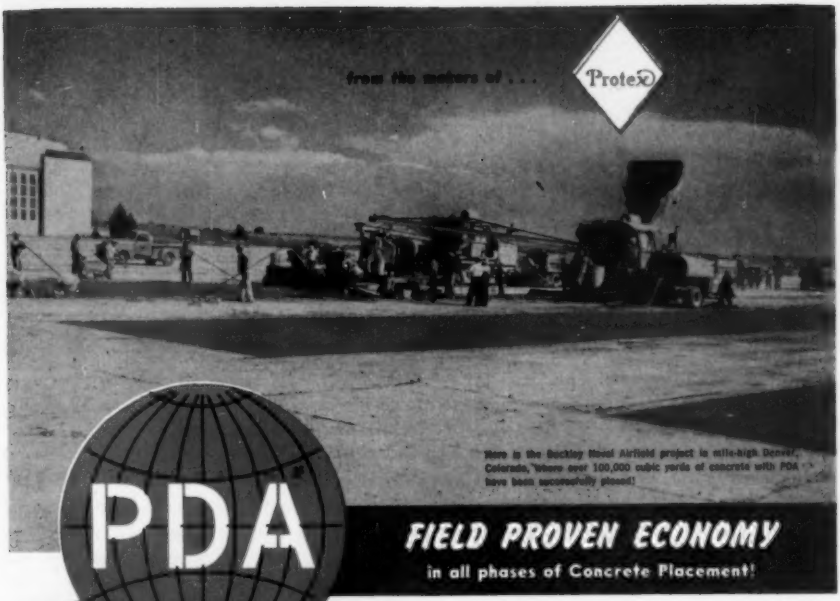
As highways have been modernized, it has become common to combine resurfacing and widening, a practice which offers definite advantages. Probably most important is that by constructing the widening integral with the resurfacing it prevents the thin edge of the resurfacing from separating and becoming unsupported by the edge of the old slab.

Structural Design Session with ASCE

The session on structural design, one of two concurrent sessions on Wednesday afternoon, February 26, was a joint session with the ASCE Structural Division. Topics on concrete structural design of interest to members of both societies were featured. Prof. Phil M. Ferguson, department of civil engineering, University of Texas, Austin, who is vice-president of ACI and chairman of the ASCE Committee on Masonry and Reinforced Concrete, presided. The reports are summarized below.

Influence of Design and Details on Concrete Deterioration—P. D. MIESENHELDER, concrete research engineer, Operations and Maintenance Department, Engineering Division, Association of American Railroads, Chicago

This paper dealt with deterioration of concrete which is influenced by or is a consequence of design features of the structure. When deterioration of concrete is observed on a structure, it is often considered a weakness or shortcoming of the material. However, this conclusion is confusing when it is observed that other portions of the same structure are in excellent condition and especially when it is obvious that the two portions were of the same materials and placed at the same time. As more and more structures are examined, it becomes apparent that there is a similarity of conditions existing where deterioration is evident. Primarily the deterioration is the result of freezing and thawing. The difference between the affected and unaffected parts of a structure results from the amount of water absorbed by the concrete and the difference in absorption results from features of the structure. Where water is continuously available to the concrete for too long a time, such as results from inadequate or improper drainage, high absorption occurs. As the water freezes it expands and if confined the force developed by the expansion is of a magnitude much greater than the strength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Here is the Buckley Road Airfield project in mile-high Denver, Colorado, where over 100,000 cubic yards of concrete with PDA have been successfully placed!

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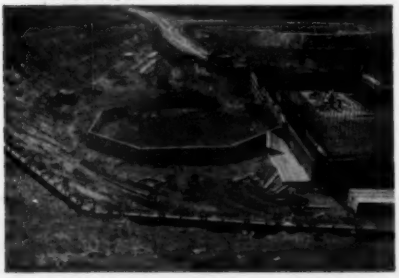

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* T.M. Reg.

Douglas E. Parsons, left, and Stanton S. Walker, center, were elected to honorary membership in the Institute by unanimous vote of the Board of Direction. Joe W. Kelly, standing at the right, is ACI's new vice-president. Eugene Freyssinet, prominent French authority on prestressing, also elected an honorary member, could not be present

of the materials. This type of deterioration was illustrated with numerous examples.

Effect of Axial Compression on Shear Strength of Reinforced Concrete Frame Members—J. W. BALDWIN, JR., instructor of theoretical and applied mechanics, University of Illinois, Urbana, and I. M. VIEST, bridge research engineer, AASHO Road Test, Ottawa, Ill.

An experimental investigation was conducted to determine the effect of axial compression on the shear strength of reinforced concrete members without web reinforcement. It involved tests of knee frames with ratios of axial force to shear varying from zero to six, and covered the entire range from failure caused by shear in the absence of axial load to failure caused by eccentric compression. The investigation was an extension of an earlier study involving specimens with the axial load equal to shear and specimens with no axial load. Inamuch as the earlier study provided an extensive coverage of the shear span, concrete strength, and percentage of steel, the extension reported in this paper was limited almost entirely to variations of the load ratio.

Observed diagonal tension cracking loads were found in good agreement with the results of the earlier study. On the other hand, the shear compression strength was found to increase with axial load considerably faster than indicated by the earlier tests. A modification of an empirical parameter in an existing theoretical expression for the ultimate strength was found necessary. This modification suggests that the compatibility of strains, as well as the equilibrium of forces, is a function of the axial load.

Ultimate Shear Strength of Reinforced Concrete Members without Shear Reinforcement—CHARLES S. WHITNEY, partner, Ammann & Whitney, Consulting Engineers, New York and Milwaukee

The method for estimating shear strength proposed in this paper is radically different from that currently in use, but it appears to be well supported by the results of tests which have covered a wide range of proportions and concrete and steel strengths.

The conventional shear formula

$$v = V/bjd = kf_c'$$

is not suitable for use because the shear strength is not a simple function of concrete strength, but depends largely on the amount of flexural reinforcement and its efficiency. Also tests show that because of bond failure due to splitting of concrete, the flexural reinforcement cannot be fully effective if it is too closely spaced.

(This report was published in October 1957 ACI JOURNAL, p. 265.)

Development of Precast Concrete Girders Reinforced with High-Strength Deformed Bars — J. R. GASTON and EIVIND HOGNESTAD, respectively, assistant development engineer and manager, Structural Development Department, Portland Cement Association, Chicago

Two 0.38-scale model roof girders were tested to develop an unusual type of precast concrete building frame. The roof girder selected departs from customary practice primarily by its slender and graceful T-section, by its high-strength longitudinal tension reinforcement, and by its inclined stirrup reinforcement. Structural design was based on the ultimate strength design procedure given in the appendix of the 1956 ACI Building Code, with some departures justified by the model girder test results. Twenty 58-ft girders were later manufactured for two laboratory buildings.

Research Index Has International Coverage
81 organizations report research in progress

The assignment of Committee 115, Research, "is to review and correlate research in concrete and reinforced concrete and to consider research methods and objectives." The committee has attempted to fulfill this assignment through the research sessions at the annual conventions, through the distribution of the annual compilation of research projects at the various laboratories, and through limited correspondence among committee members.

The 1958 compilation is divided into three parts. The first part lists 39 educational institutions that are carrying out concrete research and gives the titles of the projects that are underway. The second part provides similar information for 42 non-educational organizations. In the third part the projects are classified under subject headings. Organizations from all parts of the United States and from several foreign countries reported to the committee. These organizations included universities, highway departments, federal departments, cement companies, industrial laboratories, commercial laboratories, and various associations. Many phases of research in the fields of aggregates, admixtures, cements, and plain and reinforced concrete were reported. If you did not secure a copy of the compilation at the convention, you may do so by writing ACI headquarters.

Eleven excellent reports were presented at the research session in Chicago. Following are synopses as provided by program participants.

Pumpability and Other Physical Characteristics of Sand-Cement Grout by Thomas B. Kennedy: The pumpability and other physical characteristics of portland-cement grout adulterated with sand from which grains larger than No. 16 sieve had been removed has been investigated.

The investigation has shown that two parts of sand to one of cement by weight can be pumped regardless of grading or type of sand. The amount of sand that can be utilized per unit volume of grout appears to be a function of the amount of material in the grout finer than the No. 100 sieve. The use of fine materials, such as diatomite, fly ash, etc., were found to be beneficial.

Ionic Barrier Moisture Meter for Measurement of Moisture Content and Moisture Distribution in Hardened Concrete by Alexander Klein: In the present stage of development the meter consists essentially of a hygrostat utilizing electrical measurements and enveloped by an ionic-barrier membrane. The membrane consists of two sheets of ion-exchange resins, one an anionic permselective membrane, the other a cationic permselective membrane.

When placed in concrete, the calibrated meter changes in impedance with change in moisture content. Inasmuch as the ionic barrier prevents movement of salts to the

hygrostat, the electrical response of the meter is independent of composition of concrete, regardless of concentration or nature of salts in solution.

Accelerated Shrinkage Test of Concrete Masonry Units by David Watstein: A series of tests was carried out at the National Bureau of Standards laboratories to develop a rapid and reproducible test procedure for determining drying shrinkage of concrete masonry units at room temperature. The test procedure consisted essentially of measuring the shrinkage at 73 F and about 10 percent relative humidity of horizontal sections about ½ in. thick cut from 8 x 8 x 16-in. blocks. The shrinkage of these sections was nearly the same as the shrinkage of whole blocks and "remnant" blocks which remained after removal of the thin sections. The time required for completion of the tests using thin sections was approximately 7 days as compared with 40 to 60 days required for the larger units.

Tensile Fatigue Characteristics of Concrete Mortar Bars by James S. Blackmann: The Sonntag universal fatigue testing machine offers a simple and reliable method for establishing general fatigue characteristics of concrete. The data obtained indicate the characteristic shape of the fatigue curve and the effect of two different patterns of fluctuating stress on the endurance limit.

Fatigue Strength of Prestressed Concrete Beams in Flexure by Fuad S. Nuwaysir: A review of a theory, newly developed at Lehigh University, for predicting the dynamic ultimate flexural moment of prestressed concrete beams was presented. The dynamic ultimate moment is defined as the maximum bending moment which can be induced by a constant pulsating load for one million cycles without fatigue failure of the member. Full size beams that have been tested at the Fritz Engineering Laboratory under dynamic loadings were used as illustrative examples. A brief summary of the planned future research program was included.

Plastic Flow in Reinforced Concrete Beams under Repeated Loadings by Thomas E. Stelson: Twenty-five concrete beams with regular tensile reinforcement were tested under repeated loads applied 320 times per min. The elastic and plastic strains in the concrete and steel were measured at the center section on the beams. The results are compared with measurements taken from similar beams under static loads. The time rate of plastic flow was greatly increased by the application of repeated loads as compared with static loads. The total strains under repeated and static loads, however, were approximately the same.

Repeated Load Tests on Prestressed Concrete Railroad Bridges by E. J. Ruble: The discussion covered research being conducted on full-size prestressed concrete railroad bridge slabs with particular emphasis on the practical use of ½-in. strands for pretensioning. Laboratory tests on prestressed concrete beams and prestressed concrete railroad cross ties were described. Dynamic effects in prestressed concrete bridges resulting from the passage of railroad trains were compared with those in regular concrete bridges.

Shear Strength of Prestressed Concrete I-Beams with Web Reinforcement by M. A. Sozen: Tests were made on 37 simply-supported prestressed concrete I-beams with web reinforcement. The principal variables were: web thickness, longitudinal steel ratio, concrete strength, web reinforcement ratio, and length of shear span. Web reinforcement improved the behavior of beams for which the inclined cracking loads were smaller than the flexural ultimate loads. It was found that the amount of web reinforcement required to develop the flexural capacity of the beam was primarily a function of the difference between the flexural ultimate and the inclined cracking loads.

George Werner presented a report on qualification tests for retarders. Deleterious constituents of gravels and their effect on concrete durability were discussed by Robert Schuster, and Paul J. Brennan reported on triaxial compressive tests of fly ash concrete.

The committee is anxious to increase the coverage of research activities here and abroad. Anyone having suggestions or knowing of omissions in the compilation should advise the secretary of the committee. Information will not be given publicity unless permitted by the correspondent. Address inquiries to:

S. J. CHAMBERLIN
Secretary, ACI Committee 115
T. and A. M. Laboratory
Iowa State College
Ames, Iowa

McHenry Heads ACI for 1958

Douglas McHenry, director of development, Portland Cement Association, Chicago, has been elected for a 1-year term as president of the Institute, succeeding Walter H. Price, head of the engineering laboratories, U. S. Bureau of Reclamation, Denver. Joe W. Kelly, professor and vice-chairman, department of civil engineering, University of California, Berkeley, was elected to a 2-year term as vice-president. Phil M. Ferguson, department of civil engineering, University of Texas, elected at the 1957 convention to a 2-year term as vice-president, will continue in that office.

Chosen for 3-year terms as directors were: A. Amirikian, chief design engineer, Bureau of Yards and Docks, U. S. Navy, Washington, D. C.; S. J. Chamberlin, professor, theoretical and applied mechanics department, Iowa State College, Ames; Bryant Mather, chief of the special investigations branch, Concrete Division, Waterways Experiment Station, Jackson, Miss.; and Cedric Willson, vice-president and chief engineer, Texas Industries, Inc., Fort Worth. Stanton Walker, director of engineering, National Sand and Gravel Association and National Ready Mixed Concrete Association, Washington, D. C., has been appointed a director of the Institute for a 1-year term to fill a vacancy created by the election of Joe W. Kelly to the vice-presidency.

Douglas McHenry has been a member of ACI since 1936 and has just completed his second year as Institute vice-president. He is well known in engineering circles as the author of numerous articles on concrete technology and, in particular, for his work while head of the structural research section of the engineering laboratories of the U. S. Bureau of Reclamation prior to joining PCA in 1952.

Mr. McHenry attended the University of Minnesota from 1921 to 1924 and then joined the Southern California Edison Co., working on the Florence Lake and Shaver Dams. Before joining the staff of the Bureau of Reclamation in 1940, he served for 6 years with the Tennessee Valley Authority on construction of Norris and Hiwassee dams and power plants, and in research studies of the structural behavior of TVA works. He has served as chairman of the ACI Technical Activities Committee and Committee 209, and was a member of the Board of Direction in 1955.

Joe W. Kelly graduated from Purdue University in 1921 and for two years he engaged in waterworks engineering. Then, after a brief term as assistant in the testing material laboratory, Purdue University, he joined the staff of the Portland Cement Association, returning later to Purdue University as a concrete specialist of the engineer-

Convention-goers

Rear Admiral J. F. Jelley, U. S. Navy, retired, civil engineer of Colorado Springs, was on hand for Chicago sessions, as was Arthur R. Lord (right) of Palos Park, Ill. Mr. Lord is an honorary ACI member

ing extension department. He joined the University of California staff in 1931.

Professor Kelly, a 30-year member of the Institute, received the Wason Medal in 1934 for the paper "Cement Investigations for Boulder Dam with the Results up to the Age of One Year," prepared in collaboration with Raymond E. Davis, R. W. Carlson, and G. E. Troxell. In 1946 he received the ACI Construction Practice Award with Bernard D. Keatts for their paper "Two Special Methods of Restoring and Strengthening Masonry Structures." He served on the ACI Board of Direction from 1952-54 and was elected in 1956 to another term as a director. He served as author-chairman of Committee 611, Inspection of Concrete, through two editions of the *ACI Manual of Concrete Inspection.*

Nominating committee

Five members were elected to the nominating committee: George W. Washa, head, department of mechanics, University of Wisconsin, Madison; George C. Ernst, professor, department of civil engineering, University of Nebraska, Lincoln; R. E. Copeland, director of engineering, National Concrete Masonry Association, Chicago; Ben C. Gerwick, Jr., president, Ben C. Gerwick, Inc., San Francisco; and J. P. Thompson, structural engineer, Portland Cement Association, Chicago.

ACI Honors Presented

At the annual awards luncheon February 26, Walter H. Price, retiring president, presented 1958 ACI awards to Herbert J. Gilkey, Arsham Amirikian, Phil M. Ferguson, R. E. Copeland, James A. Minges, and Donald S. Wild. Election of honorary members was also announced.

Turner Medal

Herbert J. Gilkey, professor of theoretical and applied mechanics, Iowa State College, Ames, was awarded the Henry C. Turner Medal for "notable leadership in advancing the knowledge of properties of plain and reinforced concrete." Long eminent in the field of research in engineering materials, especially concrete, Professor Gilkey has written several textbooks and manuals, as well as various papers dealing with concrete engineering materials and testing methods. A past-president of the Institute, in 1939 he was awarded the Wason Medal for the "most meritorious paper" along with S. J. Chamberlin and R. W. Beal for "The Bond Between Concrete and Steel."

Professor Gilkey recently retired as head of the theoretical and applied mechanics department at Iowa State. He was elected in 1957 to honorary membership in the American Society for Testing Materials.

Lindau Award

Arsham Amirikian was awarded the Alfred E. Lindau Award "in recognition of his continued devotion to the improvement of design practice for precast reinforced concrete construction." Mr. Amirikian is chief design engineer, Bureau of Yards and Docks, Department of the Navy, Washington, D. C. Much of his work in recent years has been in thin shell precast concrete construction and design of protective structures against conventional bombs and atomic blasts. He originated and developed the modern precast concrete technique utilizing thin shell cells, ribbed panels, and hollow section rigid framing.

Mr. Amirikian has been widely honored for his outstanding contributions to the development of reinforced concrete. A frequent contributor to technical journals, he is chairman of ACI Committee 324, Precast Reinforced Concrete, Thin Sections and a director of the Institute.

Wason Medals

Phil M. Ferguson was awarded the Wason Medal for Research for his paper "Some Implications of Recent Diagonal Tension Tests" (ACI JOURNAL, August 1956). Prof. Ferguson, formerly head of civil engineering department, University of Texas, previously received the same award along with J. Neils Thompson for work reported in their 1953 paper "Diagonal Tension in T-Beams Without Stirrups." He has been active in the Institute since joining in 1930, now serves as vice-president and a member of the ACI Technical Activities Committee. He is also chairman of the ASCE Committee on Masonry and Reinforced Concrete.

Awardee

James S. Minges, Marchant and Minges, West Hartford, Conn., was present in Chicago to receive the Construction Practice Award. Co-author of prize-winning paper, Donald S. Wild, also of Marchant and Minges, was unable to come to the Chicago meeting. Their paper, "Six Stories of Prestressed Slabs Erected by Lift-Slab Methods," appeared in the February 1957 JOURNAL

R. E. Copeland received the Wason Medal for Most Meritorious Paper, for his paper "Shrinkage and Temperature Stresses in Masonry" (ACI JOURNAL, February 1957). Mr. Copeland, director of engineering for the National Concrete Masonry Association, Chicago, served in 1948 as a member of the ACI Board of Direction and worked on the Technical Activities Committee during 1947-48 and 1953-55. This is the second time that Mr. Copeland has received the award. In 1941, he and C. C. Carlson were accorded this honor for their paper "Tests of the Resistance to Rain Penetration of Walls Built of Masonry and Concrete."

Construction Practice Award

James S. Minges and Donald S. Wild are the recipients of the Construction Practice Award for their paper "Six Stories of Prestressed Slabs Erected by Lift-Slab Methods," published in the February 1957 JOURNAL. Mr. Minges is a member of the firm of Marchant and Minges, consulting engineers, in West Hartford, Conn. Mr. Wild is engineer in charge of structural development for the same firm.

Honorary members named

Honorary membership in the American Concrete Institute was conferred on three eminent men in the field of concrete.

The new honorary members are Eugene Freyssinet, Société Technique pour l'Utilisation de la Précontrainte, Paris; Douglas E. Parsons, chief, Building Technology Division, National Bureau of Standards, Washington, D. C.; and Stanton Walker, director of engineering, National Sand and Gravel Association and National Ready Mixed Concrete Association, Washington, D. C.

A full review of the outstanding accomplishments of these men appeared in the "News Letter" section of the March 1958 JOURNAL.

Chicago Committee Sponsors Special Events

The Chicago Convention Committee sponsored a number of social functions including an ACI-ASCE "Get Acquainted Party" on Monday evening, February 24. A cocktail hour preceded the committee's big event of the convention, the ACI-ASCE dinner dance, complete with floor show, held in the grand ballroom of the Sherman Hotel February 26. Nearly 100 women enjoyed Chicago's hospitality—food, fun, and fashions—while their husbands attended technical sessions.

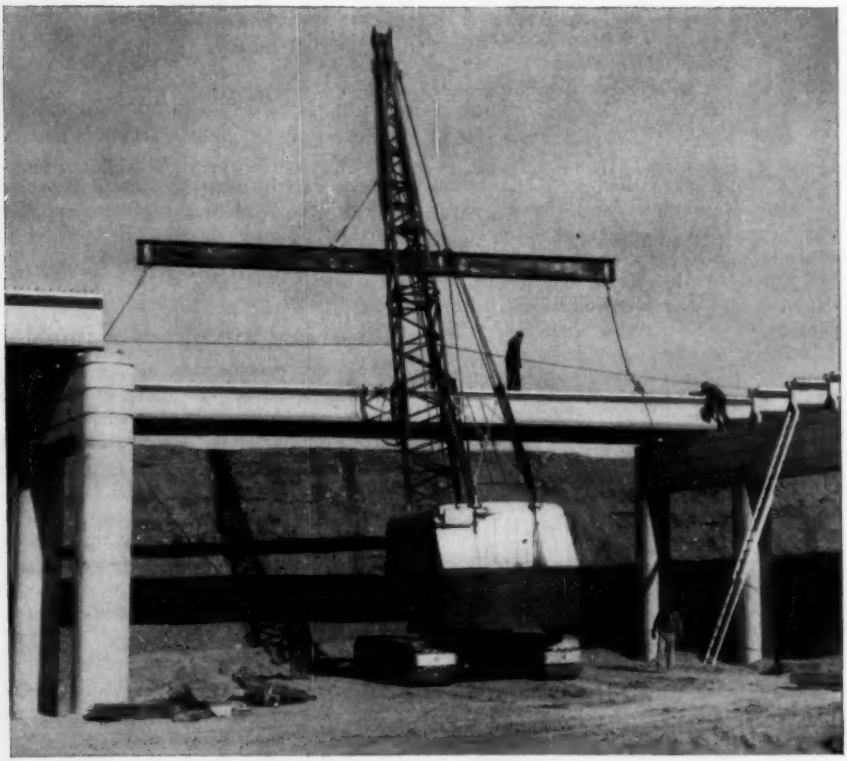
Tour of the Portland Cement Association Laboratories at nearby Skokie included inspection of main laboratories as well as the structural research and fire test buildings.

Ladies' program

Special events for ACI ladies included a fashion-luncheon in the Wedgewood Room of Marshall Field's; they visited the popular Don McNeill "Breakfast Club" and saw the

On the Garden State Parkway . . .

405 prestressed beams



PLACING MAIN SPAN GIRDERS—A crane with a spreader beam is erecting a main span girder prestressed with USS American Super-Tens Stress-Relieved Strand. Note the ease of erection.

USS **American Super-Tens**
Stress-Relieved Wire & Strand

These people built this section of the Garden State Parkway: Design Engineers: Fay, Spofford & Thorndike, Inc., Boston, Mass. • Contractor & Manufacturer of Prestressed Beams: Reid Contracting Company, Inc., Woodbridge, New Jersey; L. W. Lancaster, Chief Engineer • Consulting Engineer for Contractor: Charles C. Zollman & Associates, Newtown Square, Pennsylvania.

were produced in 81 days

Linking New Jersey with New York and New England, the recently completed section of the Garden State Parkway is 9¼ miles of ultramodern highway. Along this new super-road there are eighteen prestressed concrete bridges. USS American Super-Tens Stress-Relieved Strand was used on this modern superhighway development.

The manufacturer of the prestressed beams for these bridges produced more than 400 prestressed beams in eighty-one days.

Prestressed concrete is a proven construction material with unlimited possibilities in building roofs, floors, walls, beams, slabs, columns, and foundations, as well as for bridges. American Steel & Wire pioneered in this development in 1951 by being the *first* to develop a new product—special high tensile strength strand for the first pretensioned bridge in this country. American Steel & Wire has a large engineering staff—men with experience in the uses of prestressed concrete. They will be happy to discuss its application to any of *your* construction needs.

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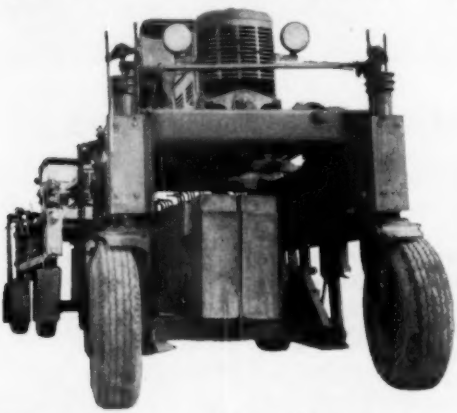

NEW WAY TO MOVE GIRDERS—Here a straddle truck is transporting a girder from the prestressing plant to the job. This was a time- and money-saving way to transport prestressed girders. Construction was speeded up because the girders were precast, then transported to the job site for immediate erection.

HERE IS the new USS American Super-Tens Stress-Relieved Strand reel furnished in standard sizes of 50" and 44". The new 2-ply construction resists warping and splitting, resulting in longer service life.

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United States Steel Export Company, Distributors Abroad

world-famous puppet show at the Kungsholm restaurant. The puppet opera was "Tosca," and a hot smorgasbord luncheon was served on the Opera Terrace. Informal coffee hours and escorted shopping tours were offered daily. Countess Maria Pulaski gave a stirring account of her espionage adventures during World War II at the afternoon tea, February 24, at the Morrison Hotel.

Chicago convention committee

The general committee on arrangements for the Chicago gathering was headed by A. Allan Bates, vice-president, research and development, Portland Cement Association, Chicago; Jack R. Janney, consulting engineer, Glenview, Ill., was secretary; George F. Long, Inland Steel Co., Chicago, was treasurer.

Subcommittee chairmen were: H. C. Delzell, Concrete Reinforcing Steel Institute, in charge of entertainment; Mrs. A. Allan Bates, hostess; W. W. Wallace, Portland Cement Association, publicity; and Earl J. Beard, Department of Public Works, City of Chicago, inspection tours.

Serving as an advisory committee were: William Armstrong, Material Service Corp.; Craig J. Cain, Chicago Fly Ash Co.; A. P. Christensen, Sumner Sollitt Co.; R. H. Clore, Universal Atlas Cement Co.; and E. M. Fucik, Harza Engineering Co. Other members of the advisory committee were: O. W. Irwin of the Rail Steel Bar Association, Jack R. Janney, consulting engineer; W. P. Markert, National Concrete Masonry Association; also Douglas McHenry, Portland Cement Association; Carl E. Wuerpel, Marquette Cement Mfg. Co.; and H. F. Peckworth, American Concrete Pipe Association, who acted as liaison between ACI and ASCE.

Mrs. A. Allan Bates served as chairman of the Ladies' Committee assisted by the following:

Mrs. William Avery
Mrs. C. C. Carlson
Mrs. Miles D. Catton
Mrs. L. D. Childs
Mrs. R. E. Copeland
Mrs. A. M. Davis
Mrs. R. C. Elstner
Mrs. E. J. Felt
Mrs. J. R. Gaston
Mrs. N. W. Hanson
Mrs. Eivind Hognestad
Mrs. Jack R. Janney
Mrs. P. H. Kaar
Mrs. Paul Klieger
Mrs. Alan Mattock
Mrs. Douglas McHenry
Mrs. G. E. Monfore
Mrs. C. C. Oleson
Mrs. George Paris
Mrs. T. C. Powers
Mrs. Paul Rogers
Mrs. J. J. Shideler
Mrs. I. L. Tyler
Mrs. Hubert Woods
Mrs. C. E. Wuerpel

Henry L. Kennedy Award established

The Board of Direction of the American Concrete Institute, at its February 23 meeting in Chicago, approved the establishment of an award in honor of the late Henry L. Kennedy, past president of the Institute who headed the building committee at the time of his death. Following is the Board's resolution:

> "This award, founded in 1958, shall be given only for outstanding technical or administrative service to the Institute and is not mandatory each year. The bases for selection of awardees shall be outstanding activity or service that has enhanced the Institute's prestige, marked leadership in technical, administrative, or special committee work, or other distinguished service to the Institute.
>
> "Except under unusual circumstances, honorary members, past presidents, and current officers and directors shall not be considered for this award. The award shall be in the form of a framed scroll, the wording of which shall be as determined by the Board of Direction."

Mr. Kennedy was manager of the construction specialties department of Dewey and Almy Chemical Co., Cambridge, Mass., at the time of his death. An internationally known authority on concrete, he pioneered the use of grinding aids for the efficient manufacture of portland cement, and air-entraining agents to improve concrete durability.

An ACI member for 23 years, Mr. Kennedy contributed frequent papers for publication in the JOURNAL and devoted much time and effort to committee and administrative work. He was president of the Institute in 1953, and past-president member of the Board of Direction in 1954-56. He served as Institute vice-president in 1951-52 and was a director in 1944-49. As head of the ACI Building Committee he took the lead in planning and fund raising activity for the ACI headquarters building recently completed in Detroit.

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ACI Technical Activities Committee for 1958

Walter J. McCoy, Lehigh Portland Cement Co., Coplay, Pa., has been reappointed chairman of the Technical Activities Committee for 1958. ACI president Douglas McHenry, Portland Cement Association, Chicago, becomes an ex-officio member of the group.

Michael V. Pregnoff, structural engineer, Pregnoff and Matheu, San Francisco, was reappointed to a 2-year term. G. E. Burnett, engineer, U. S. Bureau of Reclamation, Denver, and E. A. Finney, director, Research Laboratory Michigan State Highway Department, East Lansing, were newly appointed to the committee for 2-year terms.

Remaining members of the group include Clyde E. Kesler, University of Illinois, Urbana; Bryant Mather, Waterways Experiment Station, Jackson, Miss.; Lewis H. Tuthill, California State Department of Water Resources, Sacramento; and William A. Maples, secretary-treasurer of ACI, who will continue as secretary of the Technical Activities Committee.

Seaboard personnel changes

T. B. Hutcheson has been appointed chief engineer of the Seaboard Air Line Railroad Co., Norfolk, Va., succeeding W. D. Simpson, who recently retired after 42 years in the engineering department of the company. J. T. Ward has been appointed assistant chief engineer.

20% Discount

Bound Volume 55

of ACI Proceedings will cost only $6 if paid for before June 15. Regular member price is $7.50. Advance order price to nonmember subscribers is $7.20; to nonmembers who are not subscribers, $16.80; regular prices are $9 and $21, to the latter two respectively.

ACI Publications Department

New ACI Committee 623 will study foamed concretes

The Board of Direction has approved the formation of a new technical committee—Committee 623, Specifications and Practice for Foamed Concretes. R. C. Valore, Jr., director, research and development, concrete division, Texas Industries, Inc., Dallas, has been appointed chairman of the committee.

The committee will prepare recommendations for the use of foamed concretes. Specifications for materials and construction upon which such recommendations are based will be included. If necessary, establishment by ASTM of special tests for compliance with the specifications may be suggested for samples; tests for in-place construction would be recommended by the committee itself.

Data on performance, construction practice, and design practice will be collected and evaluated. Research data including the report "Cellular Concretes," ACI JOURNAL, May-June 1954, (*Proc.* V. 50, pp. 773-796 and pp. 817-836), and later research results will be reviewed by the committee.

Building Committee Report

as of February 28, 1958

Most recent report of the ACI Building Committee shows the following contributions to the Building Fund, arranged by categories.

Cement producers	$ 76,802.00
Reinforcing steel industry	40,000.00
Engineers and architects in private practice	34,206.22
Contractors	19,470.00
Ready-mixed concrete and aggregate industry	12,548.87
Membership at large	9,129.33
Admixtures manufacturers	7,526.83
Concrete machinery and specialty products	3,585.00
Concrete products manufacturers	2,595.00
Special contributions	1,766.43
Eastern Canada	1,496.90
	$ 209,126.58
Pledges in hand	2,760.00
GRAND TOTAL	$ 211,886.58

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advantages

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2. Recovers more than 80% of original thickness after compression
3. Low moisture absorption
4. Readily handled without breakage
5. Least expensive non-extruding type

recommended uses A general purpose joint filler, particularly for highway work where the top of the joint can be thoroughly sealed with Para-Plastic.

ASPHALT

A composition of asphalt, vegetable fibre and a small percentage of finely divided mineral filler, formed between two sheets of asphalt saturated paper.

advantages

1. Forms an easily compressible cushion
2. Is highly waterproof
3. Low in cost

recommended uses For interior concrete floor construction where black color is not objectionable, and most extensively in the formation of contraction joints in concrete construction where a ¼" or ⅜" thickness is often used.

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Cane Fiber Joint Filler is a non-extruding, resilient bituminous material consisting of cellular cane fibers securely bound together and uniformly saturated with a suitable bituminous binder.

advantages

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2. Ordinary carpenter saw can be used for cutting
3. Lowest cost non-extruding type
4. Recovers more than 70% of original thickness after compression

recommended uses An all-purpose joint filler extensively used for highway work where the upper portion of the joint can be thoroughly sealed with Para-Plastic Bituminous Rubber sealing compound. The complete line of Servicised Joint Fillers is described in the Servicised Catalog. Write for your copy.

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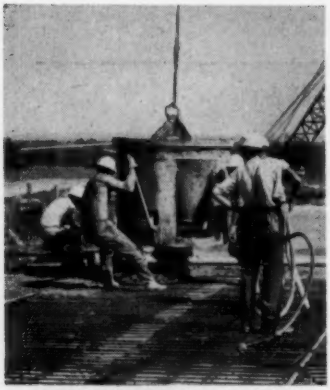

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Positions and Projects

Appointments to ACI Standards Committee

Past president Frank Kerekes, dean of the faculty, Michigan College of Mines and Technology, Houghton, has been reappointed chairman of the ACI Standards Committee for a 1-year term. Douglas E. Parsons, chief, Building Technology Division, National Bureau of Standards, Washington, D. C., will continue as vice-chairman of the committee.

Newly appointed members for 3-year terms include Joe W. Kelly, professor of civil engineering, University of California, Berkeley, and James A. McCarthy, professor of civil engineering, University of Notre Dame, Notre Dame, Ind.

Other members of the committee are Douglas McHenry, Portland Cement Association, Chicago; C. H. Scholer, Kansas State College, Manhattan; and C. S. Whitney, Ammann & Whitney, Milwaukee and New York.

Fourth annual Mississippi Highway Conference

The fourth annual Mississippi Highway Conference was conducted Feb. 20-21, 1958, on the University of Mississippi campus. Panel discussion on bridges conducted at one of the February 21 sessions covered three main categories: prestressed piles and beams; construction methods for hollow box girders; and hydraulics as related to highway structures.

Sponsoring organizations for the conference were Mississippi State Highway Department, Mississippi Society of Professional Engineers, Mississippi Road Builders' Association, and the School of Engineering, University of Mississippi.

Universal Atlas appoints Corcoran to new post

Robert M. Corcoran has been appointed to the newly-created position of traffic manager, eastern region, for Universal Atlas Cement Co., New York.

Stanton Walker addresses Detroit meeting

Stanton Walker, director of engineering, National Sand and Gravel Association and National Ready Mixed Concrete Association, Washington, D. C., addressed the Detroit chapter of the Construction Specifications Institute, Feb. 4, 1958, in a program presented by the Concrete Improvement Board of Detroit. Mr. Walker spoke on concrete specifications and their administration.

The next speaker, R. S. (Mike) Evans, assistant secretary, American Aggregates Corp., discussed the problems and considerations related to concrete aggregates. Ben Maibach, Jr., executive vice-president of Barton-Malow Co., general contractors, spoke on specifications from a contractors point of view. Jack Cooper, president of Cooper Supply Co., Detroit, was toastmaster for the program.

Vandegrift appointed to ACI Committee 714

Louis E. Vandegrift, professor, civil engineering department, Ohio State University, Columbus, has accepted appointment to membership on ACI Committee 714, Concrete Bins and Silos.

Lehigh opens Miami office

Lehigh Portland Cement Co. has opened a district sales office in Miami. William A. Mercer, formerly of the Lehigh Minneapolis sales office, will serve as district sales manager for the Miami district with R. J. Siegfried, formerly of the Birmingham office, acting as his assistant.

Ayres appointed Haller vice-president

Elliot A. Haller, president, Haller Testing Laboratories, Inc., New York, has announced the appointment of Raymond A. Ayres as vice-president of the organization. Prior to joining the Haller laboratories in 1952, Mr. Ayres was division sales manager of Lehigh Materials Co., New York.

Arthur P. Clark

Acting at its February meeting in Chicago, the Institute's board of direction adopted the following resolution:

The board of direction of the American Concrete Institute notes with great regret the death on Feb. 14, 1958, of Arthur P. Clark, former director of the American Concrete Institute and noted contributor to the study of bond between reinforcing bars and concrete. His contributions to progress in the field of reinforced concrete were many and his counsel will be sorely missed by those in this field.

Author of many JOURNAL papers, Mr. Clark joined ACI in 1924. He was a member of the board of direction 1953-55 and had been active in ACI technical committees for many years, particularly ACI Committee 208, Bond Stress, and ACI-ASCE Committee 326, Shear and Diagonal Tension.

After graduating from the University of Michigan in 1903 with a BS in civil engineering, he spent 3½ years in the bridge department of the Pere Marquette Railroad. Mr. Clark had been associated with reinforced concrete engineering and reinforcement distribution studies since 1906, first with the Corrugated Bar Co. of St. Louis. He continued with that company when it consolidated with Kalman Steel Co. and was later purchased by Bethlehem Steel Co. He retired from Bethlehem in 1944 and became research associate for the American Iron and Steel Institute at the National Bureau of Standards, Washington, D. C. His work there continued until last year.

Lester C. Smull

Lester C. Smull, vice-president of Riverside Cement Co., died recently in Riverside, Calif. He had been associated with Riverside Cement Co. for more than 35 years and had been vice-president and works manager for the past 8 years.

During World War I, Mr. Smull served with the U. S. Army Signal Corps. He graduated from the University of Washington and was a member of ACI and a number of other technical societies. Last year he was appointed to the advisory committee of the San Bernardino Flood Control District and to the Air Pollution Industrial Committee of San Bernardino County.

Law testing laboratory renamed

Law Engineering Testing Co. is the new name of Law-Barrow-Agee Laboratories, Inc., one of the South's pioneer firms in the testing field. In announcing the name change, the Atlanta company also reported that Thomas C. Law, widely known in professional circles for a half-century, is vacating the presidency but will remain active in the firm as chairman of the board.

George H. Nelson, formerly executive vice-president of the firm, has been named president. A 1923 graduate of the University of Illinois, Mr. Nelson has been in the independent laboratory field for 24 years. He has had a number of technical papers published on chemical and civil engineering and highway research subjects; he was co-author with Otto C. Frei of the paper "Lightweight Structural Concrete Proportioning and Control" which appeared in the January 1958 JOURNAL.

Mr. Nelson is a member of a number of professional organizations and particularly active in ACI committee work. At the present he is chairman of ACI Committee 401, Specifications for Structural Concrete, and a member of Committee 605, Hot Weather Concreting, and Committee 613, Recommended Practice for Proportioning Concrete Mixes.

Errata

The following correction should be made in "Ultimate Resisting Moment of Beams with Compression Reinforcement," by Eugene Guillard, in the March 1958 JOURNAL:

p. 762—In Fig. 2, upper right hand corner, equation should read

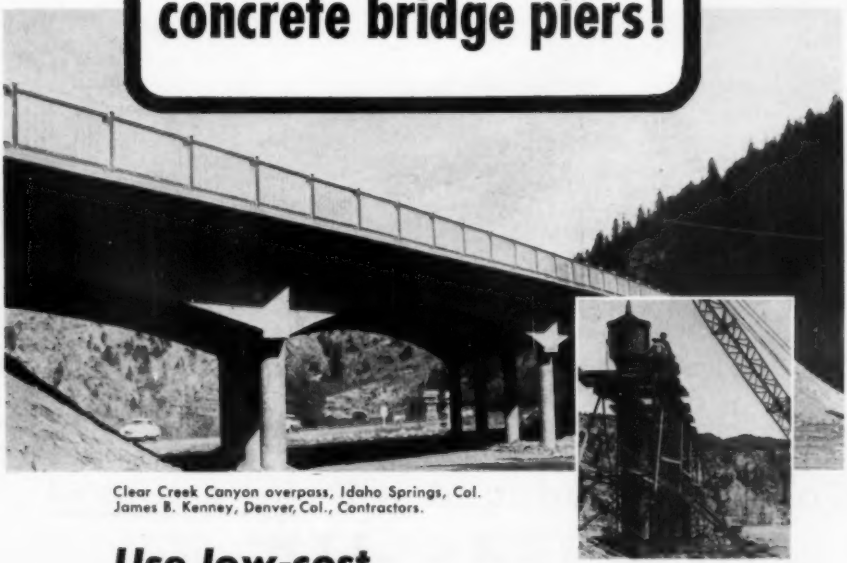
$$\frac{a}{2} C_s = A_s' f_y$$

* * *

The following correction should be made in discussion of "Under-Reinforced Concrete Beams Under Long-Term Loads," by Stefan Soretz (discusser), in the March 1958 JOURNAL:

p. 779—Fourth paragraph, line 6, should read ". . . temperature of 20 ± 0.5 C . . ."

Hold down cost of concrete bridge piers!



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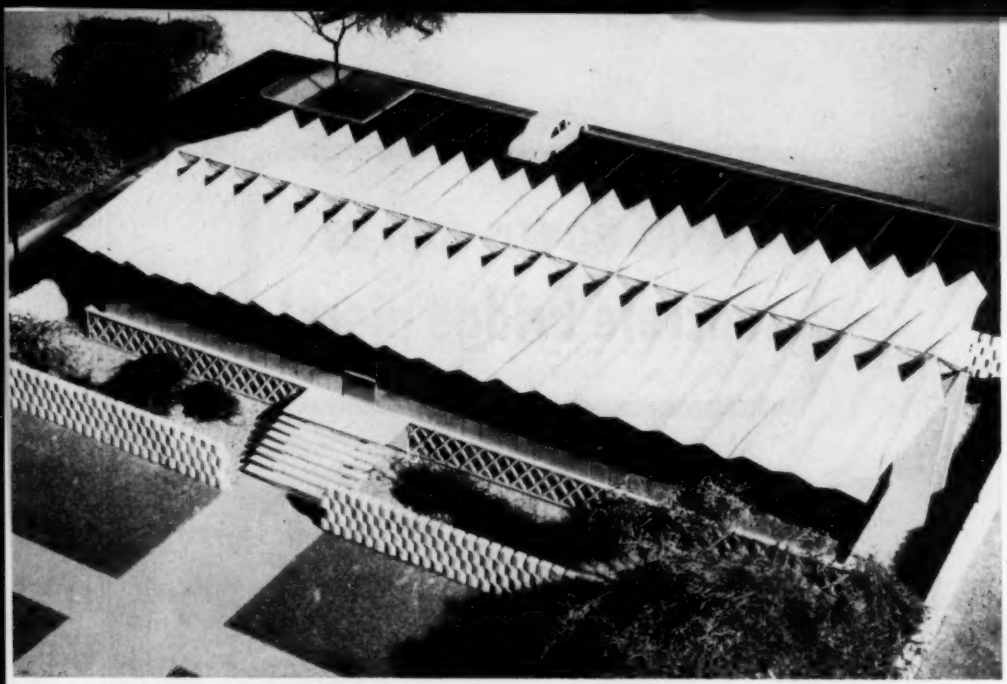

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Who's Who This Month

Walter H. Price

Retiring president Walter H. Price reviews Institute activities in "ACI's Decade of Progress" appearing on p. 825. This was originally presented by Mr. Price at the 54th annual convention in Chicago, February 26, when he concluded his year as Institute president. Mr. Price, well known to JOURNAL readers for his many ACI activities, is head of the engineering laboratories, U. S. Bureau of Reclamation, Denver.

Douglas McHenry and Joseph Karni

"Strength of Concrete Under Combined Tensile and Compressive Stresses" on p. 829 is the joint effort of Douglas McHenry and Joseph Karni.

Douglas McHenry, director of development, Portland Cement Association, Chicago, is well known in engineering circles as the author of numerous articles on concrete technology and, in particular, for his work while head of the structural research section of the engineering laboratories of the U. S. Bureau of Reclamation prior to joining PCA in 1952. As newly elected president of the Institute, he has had his many activities recently chronicled for JOURNAL readers.

Joseph Karni is head of the department of building operation research at the Building Research Station of Technion—Israel Institute of Technology, and administrative deputy to head of station at Haifa. Mr. Karni is also senior lecturer in building materials in the faculty of architecture.

Mr. Karni graduated in civil engineering from the Israel Institute of Technology in 1937, and has been a member of its staff since 1946. In 1953 he came to the United States on a scholarship from the Hebrew Technical Institute for one year of advanced studies. He worked in the development division of the Portland Cement Association doing research on combined tensile and compressive stresses and attended post-graduate courses at Northwestern University.

C. M. Wakeman, E. V. Dockweiler, H. E. Stover, and L. L. Whiteneck

Four harbor engineers having successfully used concrete in sea water construction throughout a period of more than 35 years, present their experiences and opinions in "Use of Concrete in Marine Environments" on p. 841.

Carrol M. Wakeman, testing engineer for the Los Angeles harbor department for over 30 years, has made the subject of concrete durability in sea water his hobby. Ample opportunity for this work was afforded because of the diverse types of concrete piles which are used in the harbor facilities, some of which date back to 1905.

Mr. Wakeman joined ACI in 1952; he is a member of ACI Committee 201, Durability of Concrete in Service, and ASTM Committee C-9. He attended the California Institute of Technology and Carnegie Institute of Technology and is currently a private consultant on marine problems.

Edward V. Dockweiler, Rear Admiral, U. S. Navy, retired, has been chief harbor engineer of the Los Angeles Harbor Department since 1955. Admiral Dockweiler remained in the armed services until 1948, holding posts of increasing responsibility at various naval shipyards, with design, construction, and repair of large vessels and shipyard facilities as his principal duty. He has had wide experience in the use of concrete for harbor construction. A graduate of the U. S. Naval Academy in Annapolis, he did post-graduate work at Massachusetts Institute of Technology, where he received a MS degree in naval construction.

Howard E. Stover is a newcomer in the field of concrete. He graduated with a BS in chemistry from Fresno State College in 1952. He has been employed by the Los Angeles Harbor Department as testing engineer since 1956.

Lawrence L. Whiteneck, vice-president and director of research and development for Plicoflex, Inc., Los Angeles, graduated from the State College of Washington in 1934.

He was employed by the Metropolitan Water District of Southern California in waterworks design and construction; later he became a design and development engineer for a gold mining company in the Philippine Islands. From 1946 to 1956, Mr. Whiteneck was employed by the Long Beach Harbor Department as senior harbor engineer in charge of testing evaluation and corrosion design.

Mr. Whiteneck is president-elect of the National Association of Corrosion Engineers, and a member of the American Society for Testing Materials, as well as local engineering societies in Southern California.

Robert Philleo

"Some Physical Properties of Concrete at High Temperatures" on p. 857 was prepared by Robert Philleo, research engineer in the Fire Research Section, Portland Cement Association, Chicago. He has been with the association since his graduation from Carnegie Institute of Technology in 1946, conducting research in air entrainment, dynamic testing, and the fire resistance of concrete.

He joined ACI in 1947 and at the present time is secretary of ACI Committee 207, Properties of Mass Concrete; chairman of Highway Research Board Committee B-3, Dynamic Tests of Concrete; member of ASTM Committee C-9, Concrete and Concrete Aggregates; and chairman of Subcommittee II-c, Elastic and Inelastic Properties of Concrete. His earlier paper "Comparison of Results of Three Methods for Determining Young's Modulus of Elasticity of Concrete" appeared in the January 1955 issue of the JOURNAL.

Michael Chi and Arthur F. Kirstein

"Flexural Cracks in Reinforced Concrete Beams," p. 865, has been prepared by Michael Chi and Arthur F. Kirstein on the basis of data accumulated by the late Arthur P. Clark at the National Bureau of Standards.

Michael Chi, structural engineer in the structural engineering and engineering mechanics sections of the National Bureau of Standards, Washington, D. C., graduated from the University of Tientsin, China; he received his MS degree from Louisiana State University. Prior to joining the Bureau of Standards, he worked on the analysis and design of reinforced and prestressed concrete structures with various leading consulting

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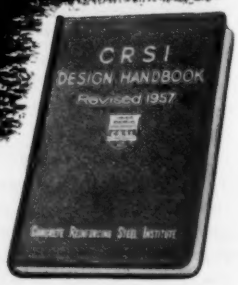

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firms on the East Coast. In his present position, Mr. Chi has been engaged in research on reinforced concrete beams and prestressed thin shells, also in the stress analysis of various metal structures. He is currently conducting a course on prestressed concrete in the civil engineering department of the Catholic University of America, Washington, D. C. He joined ACI in 1956.

Arthur F. Kirstein in his capacity as structural engineer in the structural engineering section, National Bureau of Standards, is involved in the development of criteria for predicting the types of structural failures in reinforced concrete beams. Prior to joining the NBS, Mr. Kirstein was engaged in fundamental and developmental research at the David Taylor Model Basin for 2 years. He graduated from the University of Florida in 1952; later he taught and did research work there. Mr. Kirstein has written a number of technical papers in his field. He is a member of ACI, the Society of Experimental Stress Analysis, and the American Society of Civil Engineers.

The ACI JOURNAL is still a good buy!

Rates increased Mar. 1, 1958, to $18 per year for nonmember subscribers, both foreign and domestic. ACI members continue to receive the JOURNAL as one of the considerations of dues payment.

P. G. Fluck and G. W. Washa

P. G. Fluck and G. W. Washa surveyed all available English literature on creep in preparing "Creep of Plain and Reinforced Concrete" appearing on p. 879 of this month's JOURNAL.

P. G. Fluck is a professor in the mechanics department, University of Wisconsin, Madison, where he received his PhD in 1949. He has been on the staff at the university since 1941, except for service in the U. S. Navy and short periods with the Wisconsin Highway Commission Materials Testing Laboratory and the timber mechanics division, U. S. Forest Products Laboratory.

Professor Fluck is an ACI member and associated with several other professional engineering societies; he is a member of ACI Committee 209, Volume Changes and Plastic Flow in Concrete. He was co-author with Professor Washa of "Plastic Flow (Creep) of Reinforced Concrete Continuous Beams," which was published in the January 1956 JOURNAL.

G. W. Washa, is professor and chairman of the department of mechanics at the University of Wisconsin, where he graduated in 1938. An active member of ACI since 1940, Professor Washa has been a frequent contributor to the JOURNAL. In 1940 he was awarded the Wason Medal for Research for work reported in his paper "Comparison of the Physical and Mechanical Properties of Hand Rodded and Vibrated Concrete Made with Different Cements."

Professor Washa is chairman of ACI Committee 115, Research, and a member and former chairman of Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete; he also serves on Committee 609, Compaction of Concrete by Mechanical Means, and Committee 613, Recommended Practice for Proportioning Concrete Mixes.

B. G. Singh

"Effect of the Specific Surface of Aggregates on the Compressive and Transverse Strengths of Concrete," p. 897, by B. G. Singh, is a follow-up to his article "Effect of Specific Surface of Aggregate on Consistency of

Concrete," which appeared in the April 1957 JOURNAL.

Associated with the civil engineering testing laboratory in the works and hydraulics department of Port of Spain, Trinidad, British West Indies, Mr. Singh is engaged in the testing of soils and building materials, of which cement and concrete form a prominent part.

Mr. Singh received his civil engineering degree at University College, Cardiff, in 1950. He spent the ensuing 2 years as a guest student at the university's Building Research Station. During this period he studied deterioration of portland cement in storage, and began the research on poorly graded sands which he later completed in London.

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Sundaram associated with technical college in India

P. S. Mani Sundaram, who recently joined ACI, is professor and head of the department of civil engineering at the A. C. College of Engineering and Technology at Karaikudi, India.

In publishing his name as a new member in the February JOURNAL, Professor Sundaram was incorrectly listed as being on the staff of the Nova Scotia Technical College at Halifax. Halifax is his present address, however.

Stillman joins staff of American Concrete Pipe Association

Harold W. Stillman, Jr. has joined the staff of the American Concrete Pipe Association as assistant to the managing director, Howard F. Peckworth. Mr. Stillman is a graduate of the University of Illinois. He has been associated with the U. S. Army Corps of Engineers and construction contractors; he has wide experience in civil engineering.

Covey changes firm name

Ray W. Covey, structural engineer, Detroit, announces that his son, William W. Covey, has become a member of the firm and the firm name has been changed to Ray W. Covey Associates, consulting engineers. The new firm member graduated in civil engineering from Michigan State University in 1949 and has been employed by the Covey firm since that time.

Raymond elects vice-presidents

Edwin T. Morris, Gerald O'Connor, and Brig. Gen. B. B. Talley, U. S. Army retired, have been elected vice-presidents of Raymond Concrete Pile Co., New York. All three will make their headquarters in New York.

General Talley joined the company in 1956, and has recently been in charge of construction in Brasilia, Brazil. Mr. O'Connor, with Raymond since 1929, has been chief executive officer of a number of the company's Venezuelan subsidiaries. Mr. Morris joined the firm as a field engineer in 1937, had extensive experience in African and Puerto Rican activities, and has been principal officer in a number of the company's overseas operations.

Honor Roll

January 1—March 31, 1958

Membership in ACI means access to the JOURNAL which is the combined effort of many individuals working toward increased knowledge of numerous facets of the concrete field.

The member has a two-fold opportunity in ACI. Not only may he increase his own knowledge but he may participate in furthering the Institute objective of making available to the profession new and more up-to-date research data, design procedures, and construction techniques.

Blas Lamberti....9
Phil M. Ferguson....7
Joseph J. Waddell....7
Ernest L. Spencer....5
Celso A. Carbonell....4
Ernst Maag....4
Paul A. Hansen....3
Samuel Hobbs....3
H. P. Mittet....3
Luther E. Bell....2
Vincent R. Cartelli....2
Miles N. Clair....2
William L. Collier....2
W. S. Cottingham....2
Jaime de las Casas....2
Aleck S. Evans....2
Martin J. Gutzwiller....2
Arch Hull....2
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Carl H. Koontz....2
George Kurio....2
Frank E. Legg, Jr....2
James R. Libby....2
William McGuire....2
Douglas McHenry....2
Bryant Mather....2
Frank B. May....2
H. C. Pfannkuche....2
Francesco Sardella....2
C. P. Siess....2
J. P. Thompson....2
Ellis S. Vieser....2
Tsu-ming Yang....2

Kelley elected Zonolite president

The directors of Zonolite Co., Chicago, have elected J. A. Kelley president, succeeding J. B. Myers, who recently resigned. Mr. Myers will continue his association with the company as a consultant.

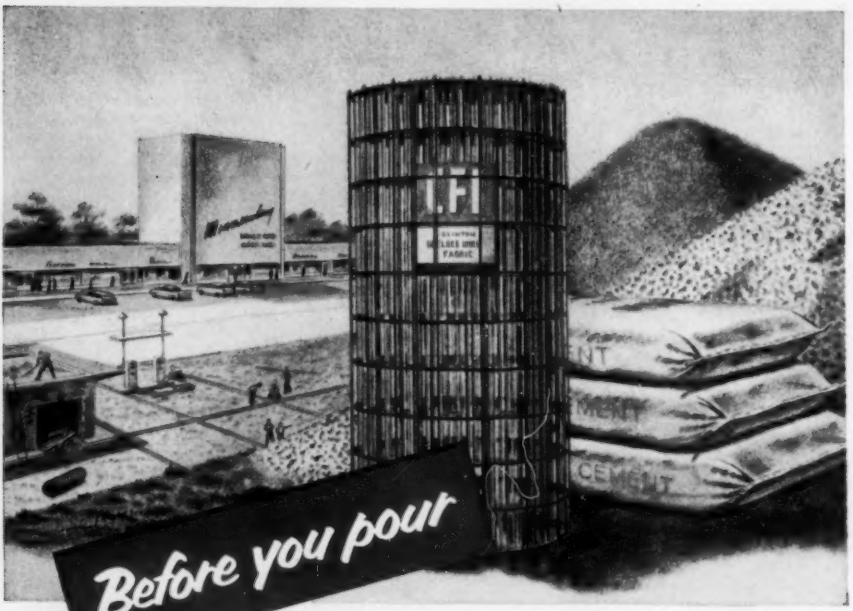
A graduate in mining engineering from the University of Pittsburgh, Mr. Kelley formerly served with the U. S. Bureau of Mines and the War Production Board. He joined Zonolite in 1946 after World War II naval service.

New Members

The Board of Direction approved 126 Individual applications, 7 Corporation, 30 Juniors, and 17 Students making a total of 180 new members. Considering losses due to resignations, deaths, and nonpayment of dues, the total membership on Mar. 1, 1958, was 9386.

Individual

ALI, MIR JAFER, Andhra Pradesh, South India (Chf. Engr., Nagarjunasagar Dam, Nalgonda Distr.)
ALVAREZ ROJAS, ALFONSO E., Uruapan, Michoacan, Mexico (Arch.)
ARONSON, LOUIS, Halifax, Nova Scotia (Mfr., Prestressed concrete products)
ATWOOD, WILLIAM, Milano, Italy (Mgr., European Div., The Master Builders Co., International Sales Div.)
AUERBACJ, ALVIN B., Corte Madera, Calif. (Lt. Col., Corps of Engrs. Presidio of San Francisco)
BARBER, ARTHUR M., La Mesa, Calif. (Civil Engrg., Design of Commercial Bldgs.)
BATTISTI, HENRY CHARLES, New York, N. Y. (C. E., Sverud, Elstad & Krueger Assocs.)
BERTHOLF, RAYMOND J., Pueblo, Colo. (Chf. Draftsman, Walter De Mordaunt)
BIXBY, HOWARD M., Washington, D. C. (Field Engr., National Crushed Stone Association)
BOUWKAMP, JACK G., Berkeley, Calif. (Asst. Prof. of Civil Engrg., Univ. of Calif.)
BROWNING, RAY A., Topeka, Kans. (Pres., Capitol Concrete Prods. Co.)
BRUNSON, H. REX, Wichita, Kans. (Engr., PCA)
BLECHSCHMIDT, FRED E., Huron, S. D. (Matls. Engr., USBR)
BUDGEN, W. E. J., London, England (Chf. Engr., L. G. Mouchel & Partners, Cons. C. E.)
BUNNELL, EDWARD R., Hammond, Ind. (Resident Engr., Hwy. & Bridges, Sverdrup & Parcel, Inc.)
BURMEISTER, CARL F., Jr., Mobile, Ala. (Arch.)
CASTILLO, LUIS F., Maracaibo, Venezuela (C. E.)
CHAUDHRY, MOHD RASHID, Minneapolis, Minn. (Civil Engrg., U of M)
COLLINS, IVAN, Jackson, Miss. (Planning & Advising Engr., Southern Bell Telephone Co.)
DEFRA, MARIO, Milan, Italy (Engr., The Master Builders Co.)
DE REGT, MARINUS, Amsterdam C, Holland (Concrete Designer, Comprimo N. V.)
DESCHAPELLE, BERNARDO, Vedado, Havana, Cuba (C. E., Jose A. Vila y Assocs.)



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LOOKING AHEAD

May 5-10, 1958—Federation Internationale de la Precontrainte, 3rd International Congress on Prestressed Concrete, Benjamin Franklin Congress Hall, Berlin, Germany.

May 20-21, 1958—Society of American Military Engineers, 38th Annual Meeting, Mayflower Hotel, Washington, D. C.

June 22-28, 1958—American Society for Testing Materials, 61st Annual Meeting and Exhibit, Hotel Statler, Boston, Mass.

June 23-27, 1958—American Society of Civil Engineers, Portland Convention, Multnomah Hotel, Portland, Ore.

Oct. 27-29, 1958—American Concrete Institute, Regional Meeting, Statler Hotel, Detroit, Mich.

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Universal Atlas promotions

George S. Neel, vice-president, central and western sales regions, Universal Atlas Cement Co., Chicago, has announced nine promotions of sales division personnel.

Frank J. Whitman has been appointed manager of sales, central region, with headquarters at Chicago; Wilbur M. Tomlinson, manager of sales, western region, with headquarters at Kansas City; Thomas E. Bertelsen, sales manager, and John W. Ettinger, assistant sales manager, both at Kansas City.

Other promotions include that of James E. Taylor to sales manager of the Illinois district with Roy D. Holloway as assistant sales manager; Auwell Fogarty, sales manager, Indiana-Michigan district with Charles E. Newton as assistant sales manager; and Harold Meyer, sales manager, Milwaukee district.

Ray succeeds Arms at PCA

Gordon K. Ray was recently appointed manager of the Highways and Municipal Bureau of the Portland Cement Association, Chicago. He succeeds Leo M. Arms, who has been named technical adviser for this bureau; Mr. Arms will retire on Dec. 1, 1958.

Mr. Ray graduated from the University of Illinois. He was with the Illinois Division of Highways for a short period prior to active duty with the Corps of Engineers, U. S. Army, from 1941 to 1945. He joined the PCA staff in 1945.

An Institute member since 1948, Mr. Arms has served in various technical and managerial positions in the PCA Highways and Municipal Bureau since 1932. A civil engineering graduate of the University of Missouri, he is a recognized authority in the field of highway engineering, and a member of ACI Committee 325, Structural Design of Concrete Pavements for Highways and Airports.

Associated General Contractors of America elect officers

Fred W. Heldenfels, Jr., a partner in the firm of Heldenfels Bros., Corpus Christi, Tex., was installed as president of the Associated General Contractors of America at their 39th annual convention in Dallas, February 10-13. James W. Cawdrey of Cawdrey and Vemo, Seattle, is the new vice-president.

Turner Construction elects three new board members

At the annual stockholders' meeting held Feb. 1, 1958, Clinton N. Hernandez, Herbert C. Mode, and George M. Reaves were elected to the board of directors of Turner Construction Co., New York. All of the newly elected directors are vice-presidents of the organization. Mr. Hernandez is in charge of the company's Chicago office; Mr. Reaves is in charge of the Boston office; and Mr. Mode is vice-president of Ferro Concrete Construction Co., Cincinnati, a Turner subsidiary.

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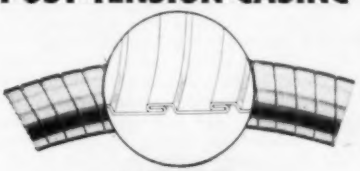

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Eduardo Torroja was a pioneer in 1933, and he is a leader in design today. He created, and directs, the world-famous Technical Institute of Construction and Cement at Costillares, Spain. He is a mathematician, and steeped in theory of structures. He also knows when to resort to experiment and model analysis. He is also a humanist, with a flair for the beautiful, as the structures in this book will reveal.

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Tools, Materials, Services

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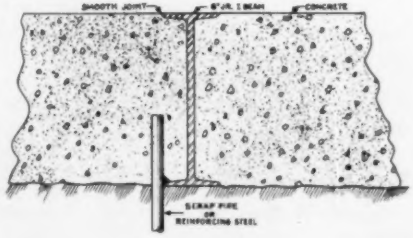
Form hardware catalog

"Form Engineering," a 60-page catalog shows clamps, tie rods, pigtail anchors, miscellaneous accessories, beam hangers, chairs for reinforcement, rubber control joints and waterstops, and rock anchors. Details and specifications are given for each item along with clearly drawn sketches showing items in use.

Catalog suggests specifications of value in producing quality concrete, and an engineering data section provides tables for use in form design.—Williams Form Engineering Corp., 1501 Madison Ave., S. E., Grand Rapids 7, Mich.

Lightweight steel beam used as expansion joint

Construction technique that uses a lightweight steel beam as an expansion joint between concrete paving sections has been successfully used by R. G. LeTourneau, noted industrial manufacturer, for buildings



and storage yards for over 20 years, according to the I-beam manufacturer. The method uses a 6-in. "junior" I-beam, a lightweight structural member made by Jones & Laughlin. The beam, weighing only 4.4 lb per linear ft, is held in position by a steel bar driven into the ground and tack-welded to the beam flange. Mr. LeTourneau states that this method permits the concrete to shrink without exposing an open space which gradually becomes filled with dirt, eventually producing buckling. He uses the small I-beam as a screed to strike off the concrete, and places adjoining slabs without having to wait for concrete to harden. —Jones and Laughlin Steel Corp., 3 Gateway Center, Pittsburgh 30, Pa.

Underwater television camera views concrete

Recently the Tennessee Valley Authority witnessed a demonstration of underwater closed circuit television at Wheeler Dam, Sheffield, Ala., in a thorough examination of the apron immediately below the dam and of the spillway underwater construction for erosion, cracks, and other possible defects.

By means of the Bludworth marine underwater television system (consisting of a camera equipped with

automatic iris, monitor, and control unit) and with the aid of special lighting, not only could the face of the dam be seen on the monitor screen on the surface vessel, but cracks and erosion were magnified two or three times normal size and were clearly observed.

The camera was operated in water about 20 ft deep during examination of the apron and ranged up to 100 ft from the control unit and monitor. The equipment was operated also at a depth of about 60 ft immediately upstream from Wheeler Dam.

The underwater camera unit can be lowered or carried down by a diver, and the monitor (or picture screen) can be located either on land or on a boat, barge, or ship. Lighting is furnished for the camera unit so that underwater conditions can be seen even in darkness. The unit, which weighs 4 lb submerged, can be operated at depths up to 180 ft. The diver need only aim the camera with instructions from above since all electronic adjustments are made at the camera control unit on the surface of the water or on land, according to the manufacturer.—Bludworth Marine Division, Kearfott Co., Inc., 1500 Main Ave., Clifton, N. J.

Aluminum truck mixer

A 7-cu yd truck mixer built by Construction Machinery Co. and Kaiser Aluminum and Chemical Corp., is made completely of aluminum except for the drive train. Exhibited at the National Ready Mixed Concrete Show in Chicago in February, this model is purely experimental. Manufacturers state that this

machine will be tested to destruction to determine how and where lightweight materials can be used effectively at a savings. Rugged tests will take several months and subject the mixer to stresses and abrasive wear it would never encounter in normal use. If these tests prove the machine to be practicable, this lighter weight mixer may offer savings in time, labor, fuel, and maintenance. The two distributing chutes shown in the picture weigh a total of 47 lb, about half the weight of steel chutes.

The regular Transcrete line has five sizes, 4 to 7 cu yd capacity, with separate engine drive and four sizes, 5 to 7 cu yd capacity, with truck engine drive. More information about the aluminum truck mixer is available in an illustrated booklet, "Exploring Tomorrow."—Construction Machinery Co., Waterloo, Iowa

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Steel scaffold shoring bulletin

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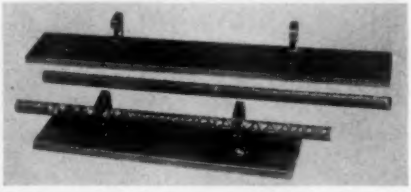
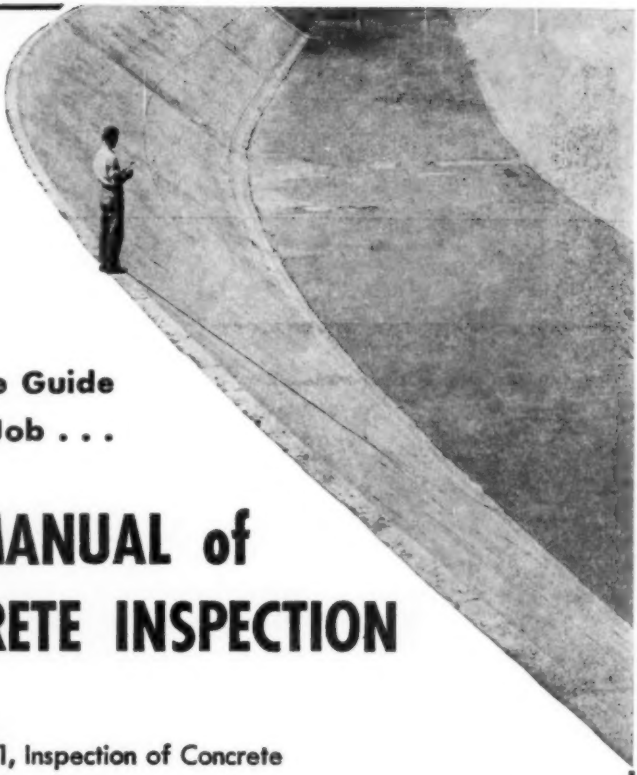
End-welded stud

Installation of reinforcing bars up to $\frac{3}{4}$ in. in diameter is secured faster and more economically with a Re-Bar stud, according to the manufacturer, who states that this end-welded stud is specifically designed



for shotcreting and other concreting applications. It eliminates both hand welding of fasteners and tying of bar. The bar is placed in a jaw on the stud and is then locked into place with a couple of hammer blows.

In standard dimensions, this stud, designated Type R4L, measures $\frac{3}{4}$ in. wide and $\frac{1}{16}$ in. thick. The recommended minimum distance from the weld base to the edge of the jaw, which is the distance the bar will be held away from the steel, is $\frac{1}{16}$ in. Studs may also be obtained with this dimension as $1\frac{1}{4}$ in., $1\frac{1}{2}$ in., $1\frac{1}{16}$ in., or $1\frac{7}{16}$ in.—Nelson Stud Welding Division, Gregory Industries, Inc., Lorain, Ohio



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